

1988

Rock slope stability studies in Siskiyou National Forest

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AN ABSTRACT OF THE THESIS OF Greg Visconty for the Master of Science in Geology presented May 20, 1988.

Title: Rock Slope Stability Studies in Siskiyou National Forest.

APPROVED BY MEMBERS OF THE THESIS COMMITTEE:


Ansel Johnson, Chairman


Marvin Beeson


Virginia Pfaff


Kent Lall

The line mapping method of Piteau and Martin (1977) was tested on two different rock type road cuts in the Siskiyou National Forest, and was found to be an efficient means of collecting geological data for rock slope stability analysis. The unbiased approach of this method calls for close scrutiny of the outcrops in question, covering more ground than other methods in less time. In turn, this close attention to every crack in the outcrop reveals more about

the stability of the slope, and can reveal hidden hazards of rock fall.

The supportive systems for analyzing the data - stereonets and computer program packages of Watts (1986) - led to the discoveries of several potential plane and wedge failures which were not initially visible. Also revealed was the fairly stable condition of the massive wedge at Elk River, which appeared to be extremely hazardous.

Each potential failure was analyzed for its Factor of Safety under dry and water saturated conditions, and the cohesion necessary to maintain stability was reported.

ROCK SLOPE STABILITY STUDIES IN
SISKIYOU NATIONAL FOREST

by

GREG VISCONTY

A thesis submitted in partial fulfillment of the
requirements for the degree of

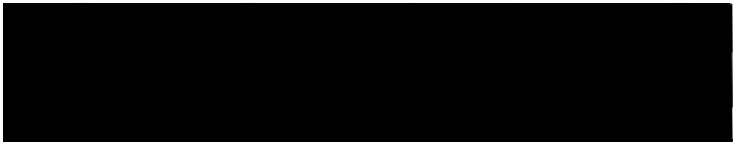
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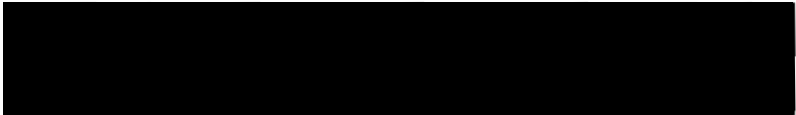
Portland State University


1988

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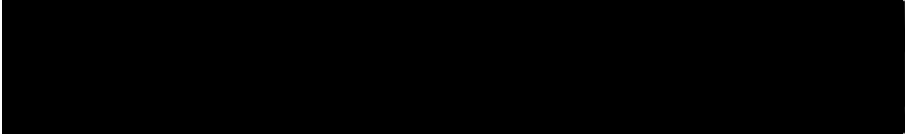

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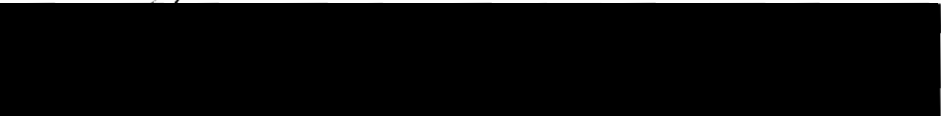

Bernard Ross, Vice Provost for Graduate Studies

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CHAPTER I

INTRODUCTION

There are several procedures used for collecting data for rock slope stability analyses. They include whole slope mapping, window mapping, cross and long sections, and line mapping. The height, attitude and accessibility of the slopes in question will dictate the best method to use. This study is concerned with the efficiency of the line mapping method of Piteau and Martin (1977) as applied to road cuts up to 50 meters in height, in conjunction with the computer assisted data analysis program package of Watts (1986). Following are brief descriptions of the mapping methods mentioned above.

Whole Slope Mapping

The ultimate goal of any study of this nature is to describe the stable and hazardous regions of an outcrop. It is best to gather all the information there is, which can only be done by mapping the entire slope area, recording rock type, physical characteristics, discontinuities, etc. This information can then be displayed on a long section of the rock slope face. These characteristics can then be projected into the rock outcrop, using any additional data acquired by drilling into the face, to best describe the

stability conditions and hazardous regions in the rock slope. Although this procedure offers the most information, it can be time consuming and costly. It can also be hazardous work depending upon the slope angle and the conditions of the face.

Window Mapping

To reduce the expense and risk of whole slope mapping, the "window" method (Hoek and Bray 1981) is used whereby a patchwork of squares is established across the face. The size and spacing of these squares is up to the geologist. Each "window" is then extensively mapped, and these structures and characteristics are projected through the unmapped areas. One danger of this system may be the tendency to map those areas most accessible, leading to a biased view of the rock slope.

A variation of this, called fracture set mapping (Hoek and Bray 1981), predetermines 6 by 12 meter bands spaced at 30 meter intervals along the outcrop, regardless of vegetation cover or access difficulties. Although this may result in a rapid accumulation of data, large expanses of the slope remain unmapped, and the final analysis must then use considerable guess work as to the conditions in these areas.

Cross Sections

With soil slope failures (planar and/or rotational

slump), simple cross sections done in the field are often adequate to properly evaluate the mode of occurrence of the failure, but they reveal only a 2-dimensional view. In rock slopes, the failures often involve 3-dimensional wedges than planes, and cross sections alone are very limited in usefulness. They can be helpful supplements to any of the mapping methods mentioned above.

Line Mapping

Line mapping involves the logging of all of the rock characteristics and discontinuities which cross a tape line stretched along the base of a rock slope. This method, described in detail in Chapter III, results in a band of geological information for the entire length of the slope. The line or lines of mapping on pre-excavated slopes may, of course, be placed anywhere on the ground, but in existing road cuts or quarry walls, the base of the slope is usually the most accessible.

CHAPTER II

BACKGROUND

In rock slope stability analysis, the field problem begins with macroscopic observations. Determining rock type in the field is usually easy enough, but microscopic evaluation may be required. A detailed mineralogical explanation of the rock may be done by the use of thin sections, but the main point of this study is the determination of the strength of the rock mass, and not necessarily its mineralogical makeup.

ROCK SLOPE CHARACTERISTICS

Weathering

The extent of weathering will affect the hardness and strength of the rock mass; it will make it softer and weaker with time. Understanding the weathering sequences of the individual rock types in a slope may aid the geologist in determining the weaker zones in the slope. Part of this stage of the analysis also involves the maintenance history of the slope, which will help indicate the size and frequency of failures to be expected.

Hardness

Even though failures in rock slopes are due almost

entirely to pre-existing, natural planes of weakness, or discontinuities such as joints and faults (Jumikis 1979), the data in an analysis should include some detail on the strength of the intact rock itself. In the case of layered sediments, for example, the underlying intact, but weak, rocks may not be able to support the overlying beds, resulting in collapse, Figure 1a, or toppling, Figure 1b.

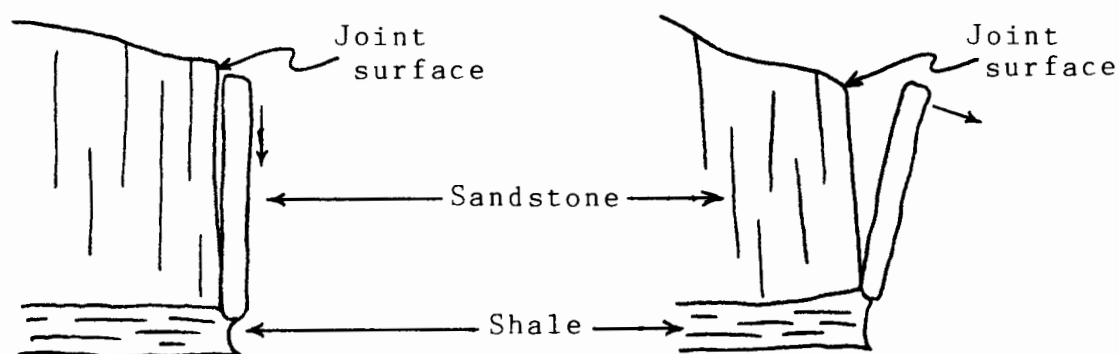


Figure 1a. Collapse. Figure 1b. Toppling.

The unconfined uniaxial compressive strength is the basic index in rock strength classification (Roberts 1977). There are numerous methods for determining this quality, most of which are performed in a laboratory on specifically shaped and cut specimens. Rapid collection of data in the field is desirable, so a quick and simple method was developed (Hoek and Bray 1981). The details of this method are shown in Tables I and II for soils and rocks, respectively, and is the method used in this study.

Discontinuities

Discontinuities are any planar breaks in the rock mass, and they may take on any of several characteristics. They encompass joints and joint sets, bedding planes, tension cracks, shear zones and faults, rock contacts, and planes such as schistosity. The attitude of these planes in the rock slope is possibly the most important aspect. The steeper the dip of the discontinuities, the less stable they tend to be. Dips between 30 and 70 degrees appear to be the most susceptible to sliding (Hoek and Bray 1981); dips less than 30 degrees usually require a force in addition to gravity to induce sliding, and dips from 70 degrees with the slope to 70 degrees against the slope usually result in toppling. The overall geometry of these planes must also be considered. Irregular or curved joints will have variable dip angles, as shown in Figure 2. The low dip angle sections may prevent the sliding of the hanging wall block, even though the steeper dips are unstable, and the shear strength of the rock mass may have to be overcome to cause failure.

The surfaces of the discontinuities need to be studied as to how conducive they are to sliding; smooth and slickensided faces are the least stable, stepped and coarse faces are the most secure. The roughness may be due simply to the nature of the protruding grains, or the the irregularity of the break.

TABLE I

DESCRIPTION FOR DETERMINING CONSISTENCY OR HARDNESS
OF SOIL USING FIELD MAPPING METHODS
(PITEAU AND MARTIN 1977)

Hardness	Consistency	Field Identification	Approximate Range of Unconfined Compressive Strength	
			Kg/cm ²	p.s.i.
S1	Very soft	Easily penetrated several inches by fist	<0.25	<3.5
S2	Soft	Easily penetrated several inches by thumb	0.25-0.5	3.5-7
S3	Firm	Can be penetrated several inches by thumb with moderate effort	0.5-1.0	7-14
S4	Stiff	Readily indented by thumb but penetrated with great effort	1.0-2.0	14-28
S5	Very stiff	Readily indented by thumbnail	2.0-4.0	28-56
S6	Hard	Indented with dif- ficulty by thumb- nail	>4.0	>56

TABLE II

DESCRIPTION FOR DETERMINING CONSISTENCY OR HARDNESS
OF ROCK USING FIELD MAPPING METHODS
(PITEAU AND MARTIN 1977)

Hardness	Consistency	Field Identification	Approximate Range of Unconfined Compressive Strength	
			Kg/cm ²	p.s.i.
R0	Extremely soft rock	Indented by thumbnail	2.0-7.0	28-100
R1	Very soft rock	Crumbles under firm blows with point of geological pick, can be peeled by a pocket knife	7.0-70	100-1,000
R2	Soft rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow of geological pick	70-280	1,000-4,000
R3	Average rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single blow of hammer end of geological pick	280-560	4,000-8,000
R4	Hard rock	Specimen requires more than one blow with hammer end of geological pick to fracture it	560-1,120	8,000-16,000
R5	Very hard rock	Specimen requires many blows of hammer end of geological pick to fracture it	1,120-2,240	16,000-32,000
R6	Extremely hard rock	Specimen can only be chipped with geological pick	>2,240	>32,000

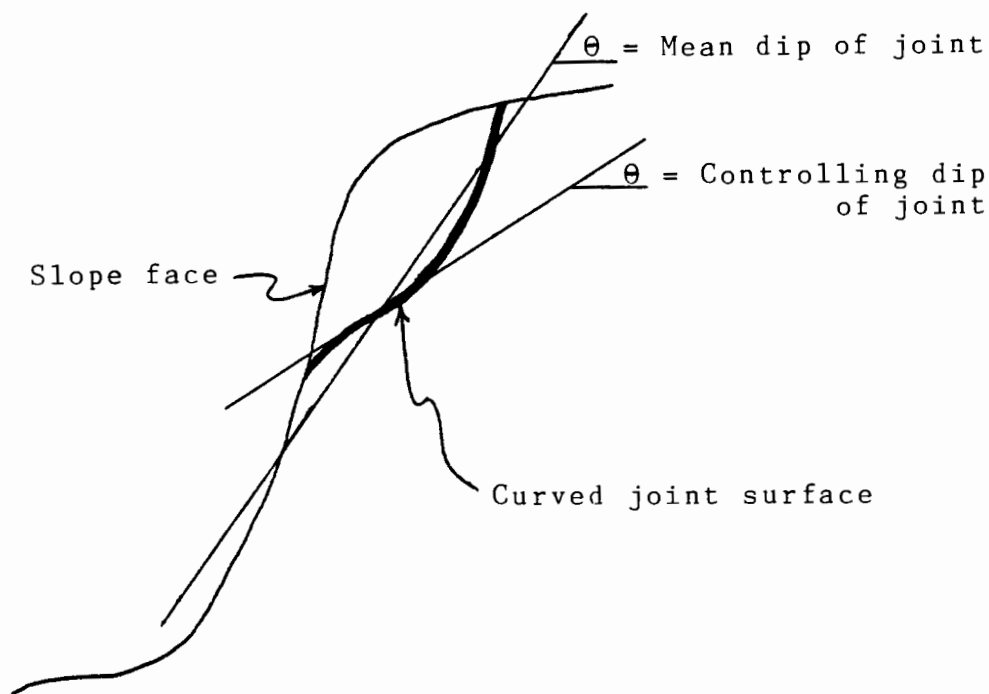


Figure 2. Curved surface of joint.

LIMITING EQUILIBRIUM

In order to understand the role of discontinuities in slope failure, a brief explanation is included on the stresses and forces which act along and across discontinuities. The following discussion is rewritten from Hoek and Bray (1981).

The properties which control the movement along any plane of fracture can be expressed in the following

equation:

$$\tau = C + \sigma_n * \tan \phi \quad (1)$$

where: τ = shear stress acting along the plane,
 σ_n = normal stress acting across the plane,
 C = cohesion bonding the two surfaces together,
 ϕ = internal angle of friction.

The relationship between the shear and normal stresses is shown graphically in Figure 3, and by a plot of the shear

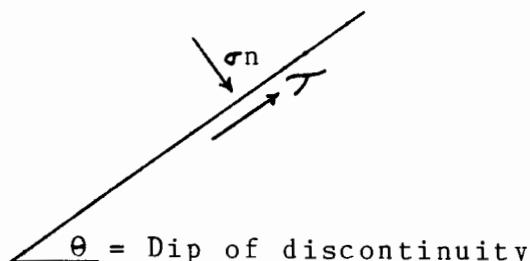


Figure 3. Shear and normal stresses.

and normal stresses, shown in Figure 4, which are necessary to just cause sliding along the discontinuity, as done in

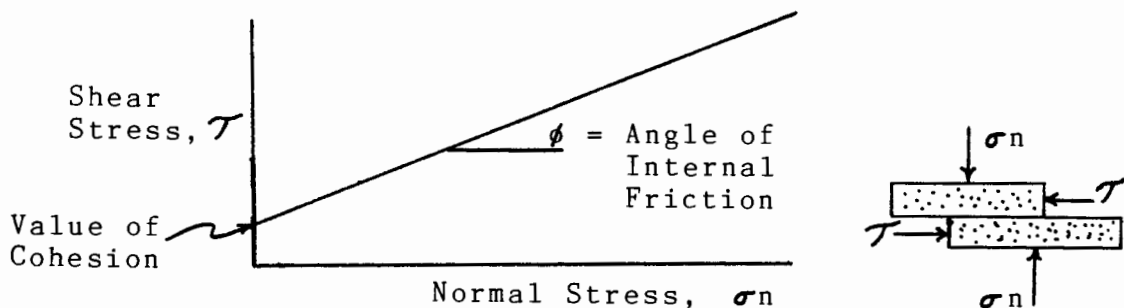


Figure 4. Shear verses normal stress.

a laboratory experiment. An increase in the normal stress on a system requires an increased shear stress to induce movement. The slope of this line represents the angle of internal friction, ϕ (Phi), not necessarily the dip angle θ (Theta), of the discontinuity at failure. The value of cohesion, C , is determined by the plot's intercept with the shear axis, and is indicative of the shear stress which is necessary for sliding failure when the normal stress on the system is zero. Put another way, cohesion is the shear strength which must be overcome to cause failure if there is no normal stress acting across the fracture plane.

Cohesion is usually limited to soils and unbroken rock masses, see Tables III and IV; fractured rocks theoretically have no cohesion as can be seen on Table III. However, the roughness of the failure surface will add to the stability of the system, and is therefore included in cohesion in stability calculations.

The units of stress are:

$$\text{Stress} = \text{Force/unit Area, in kN/m}^2 \text{ or lbs/ft}^2$$

or

$$\text{Force} = \text{Stress} * \text{unit Area, A}$$

and the forces acting on a rock system can be displayed as in Figure 5.

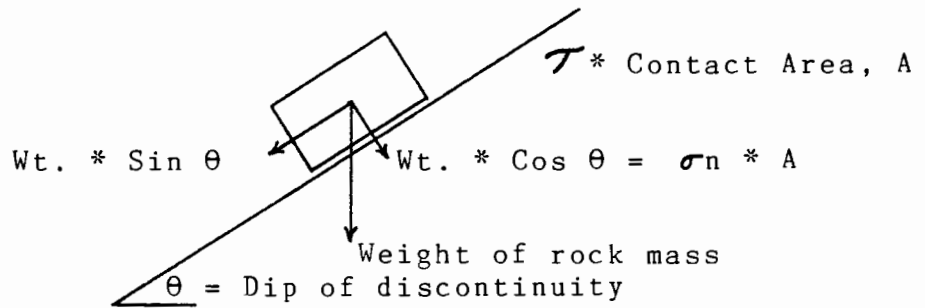


Figure 5. Forces acting on a sliding block.

The force acting normal to the plane is:

$$\sigma_n = \frac{Wt. * \cos \theta}{A} \quad (2)$$

With substitution, equation (1) becomes:

$$\tau = C + \frac{Wt. * \cos \theta}{A} * \tan \phi \quad (3)$$

The shear force which acts to stabilize this system is:

$$\tau * A = C * A + Wt. * \cos \theta * \tan \phi \quad (4)$$

and the force which acts to cause sliding is:

$$Wt. * \sin \theta \quad (5)$$

When $\tau * A = Wt. * \sin \theta$, stability exists, but the system is on the verge of failure. This condition is known as Limiting Equilibrium, and the system is referred to as

TABLE III
TYPICAL SOIL PROPERTIES
(HOEK AND BRAY 1981)

DESCRIPTION		UNIT WEIGHT		FRICTION	COHESION
TYPE	MATERIAL	(Saturated/Dry)		ANGLE	
		lb/ft3	kN/m3	ϕ	lb/ft2 kPa

COHESIONLESS					
SAND					
	Loose, uniform	118/90	19/14	28-34	
	Dense, uniform	130/109	21/17	32-40	
	Loose, mixed	124/99	20/16	34-40	
	Dense, mixed	135/116	21/18	38-46	
GRAVEL					
	Uniform	140/130	22/20	34-37	
	Mixed with sand	120/110	19/17	45-48	
BLASTED/BROKEN ROCK					
	Basalt	140/110	22/17	40-50	
	Chalk	80/62	13/10	30-40	
	Granite	125/110	20/17	45-50	
	Limestone	120/100	19/16	35-40	
	Sandstone	110/80	17/13	35-45	
	Shale	125/100	20/16	30-35	

Note: Higher friction angles in cohesionless materials occur at low confining or normal stresses.

TABLE IV
TYPICAL ROCK PROPERTIES
(HOEK AND BRAY 1981)

DESCRIPTION		UNIT WEIGHT (Saturated/Dry)		FRICTION	COHESION	
TYPE	MATERIAL	lb/ft3	kN/m3	ANGLE ϕ	lb/ft2	kPa

COHESIVE						
CLAY						
	Soft bentonite	80/30	13/6	7-13	200-400	10-20
	Very soft organic	90/40	14/6	12-16	200-600	10-30
	Soft, slightly organic	100/60	16/10	22-27	400-1000	20-50
	Soft glacial	110/76	17/12	27/32	600-1500	30-70
	Stiff glacial	130/105	20/17	30-32	1500-3000	70-150
	Glacial till mixed grain size	145/130	23/20	32-35	3000-5000	150-250
ROCK						
	Hard igneous: granite, basalt, porphyry	160-190	35-30	35-45	720000-1150000	35000-55000
	Metamorphic: quartzite, gneiss, slate	160-180	25-28	30-40	400000-800000	20000-40000
	Hard sedimentary: limestone, dolomite, sandstone	150-180	23-28	35-45	200000-600000	10000-30000
	Soft sedimentary: sandstone, coal, chalk, shale	110-150	17-23	23-35	20000-400000	1000-20000

Note: For intact rock, the unit weight of the material does not vary significantly between saturated and dry states with the exception of materials such as porous sandstones.

having a Factor of Safety (F.S.) of 1.00 by the following formula:

$$\text{Factor of Safety} = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{\tau * A}{Wt. * \sin \theta} \quad (6)$$

When F.S. = 1.00,

$$Wt. * \sin \theta = C * A + Wt. * \cos \theta * \tan \phi \quad (7)$$

If the fracture surfaces are smooth with no cohesive quality, the dip angle θ of the plane of failure which is just on the verge of sliding will be the same as the internal angle of friction ϕ , as follows:

$$Wt. * \sin \theta = (0) * A + Wt. * \cos \theta * \tan \phi \quad (8)$$

$$\frac{Wt. * \sin \theta}{Wt. * \cos \theta} = \tan \phi \quad (9)$$

therefore:

$$\theta = \phi \quad (10)$$

It is not so straight forward in the field, because relating the roughness of the discontinuity to the value of cohesion is a matter of conjecture. Determining the relationship accurately in the field is subject to experience and estimation. A simple experiment can be done in the field to arrive at an estimate of the value of ϕ , which may include the effect of cohesion upon sliding. Remove intact sections of discontinuities, such as both

contacting sides of a joint. Of course, they must be large enough to represent the structure, yet small enough to handle. Tilt the combination until the top rock mass just begins to slide, and record the angle of inclination. If there are no infilling materials, the value arrived at should be near that of documented friction angles such as shown on Table III. The rougher the surface is, the higher the angle of inclination will be. This value can then be used as an estimate of ϕ in the preliminary calculations of stability.

Infilling materials

The cohesion of a system will be affected by the presence of infilling materials deposited in the discontinuities. Mineral deposits of quartz or calcite may bond the surfaces of the discontinuity, and may cause the host rock surfaces to be out of contact with each other. In these cases, the stability characteristics are controlled by the cohesion and friction angle of the deposited minerals instead of the host rock, or by the contact of host rock and deposited minerals, as shown in Figure 6.

The presence of clays may simply increase the cohesive value of the failure plane, making it more stable. This cohesive condition may be temporary if the clays are washed out. The infilling materials may also take the form of grains of the host rock ground up during some prior

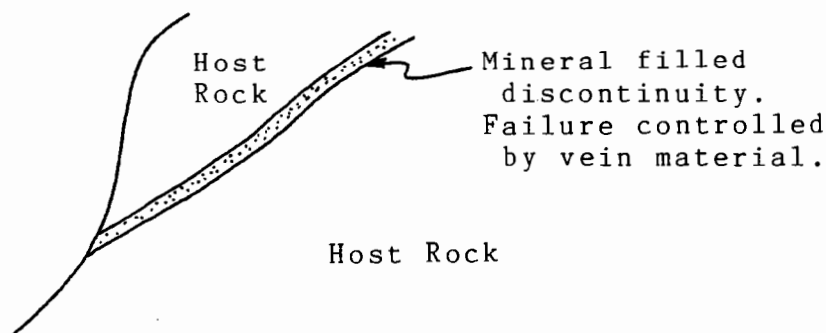


Figure 6. Infilling material in discontinuity.

movement. They may be breccia fragments which lock up the failure plane, or sand grains which act as ball bearings to greatly reduce stability.

Water conditions

The equilibrium conditions of any system will be greatly altered by the presence of water, especially water under pressure. Following is a discussion from Hoek and Bray (1981) on the effects of water pressure on discontinuities.

Confined water in the discontinuity will exert pressure on a system in all directions, including perpendicular to the fracture surfaces, denoted as "u" in Figure 7. The water pressure acts to buoy the block, which reduces the normal stabilizing pressure as follows:

$$\tau = C + (\sigma_n - u) * \tan \phi \quad (11)$$

This reduces the forces resisting sliding, and allows for a much shallower dip angle for failure. The system becomes

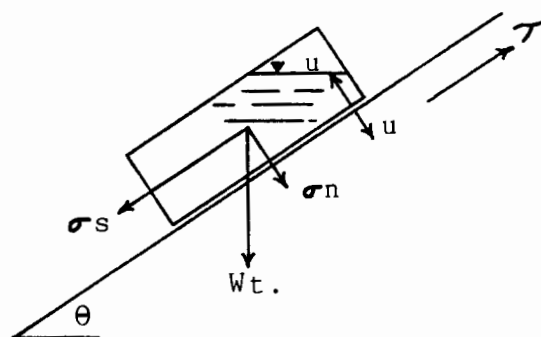


Figure 7. Water pressure on sliding block.

even more unstable when a tension crack above the potential sliding block becomes filled with free draining water. Figure 8 shows "V" as the downslope quantity of tension

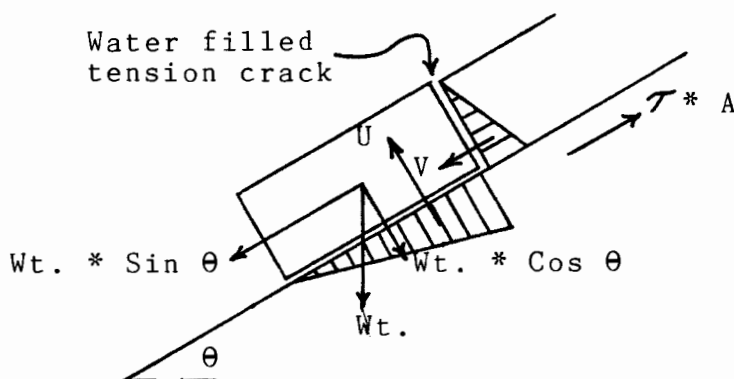


Figure 8. Water filled tension crack.

crack water force, (water pressure multiplied by the tension crack surface area), and "U" is the buoyant water force acting to float the block. The limiting equilibrium condition is then found by equation (12):

$$Wt. * (\sin \theta) + V = C * A + [Wt. * (\cos \theta) - U] * \tan \phi \quad (12)$$

Even small water pressures acting on large surface areas can

generate large forces, causing seemingly stable slopes to fail during wet seasons. Year round climatic conditions need to be considered when analyzing rock slopes, and part of the final failure prevention plan may include drains for these water filled cracks. Figure 9 shows a drain pipe installed in the bottom of a moderately dipping fault plane.



Figure 9. Drain installed in small fault.

TYPES OF FAILURES

There are four major types of failures encountered in stability analyses. Up to this point only planar failure of a sliding block has been considered. In actual rock slopes, the failure modes become far more complex, involving conditions such as intersecting joint sets, and additional

loading by man made structures.

Slump

A common type of failure in soil slopes is slump, or circular failure, shown in Figure 10. The soil material is weak with insufficient cohesion to maintain the slope angle. This condition can also exist in a rock slope which is highly fractured or extremely weathered, so much so that it behaves as a soil slope.

Plane

Plane failure, shown in Figure 11, is not common in rock slopes. The criteria are summarized below: (Hoek and Bray 1981)

1. - The strike of the failure plane should be within 20 degrees of the strike of the slope.
Otherwise, release planes are required, and failure would then be by wedging.
2. - The dip of the plane must exceed the internal angle of friction (including any additional cohesion acting on the plane).
3. - The plane must "daylight" in the slope, that is, the bottom of the plane must be exposed in the slope face.
4. - Unless the failure plane passes through a curve in the slope, it must have lateral release planes which offer little stability.

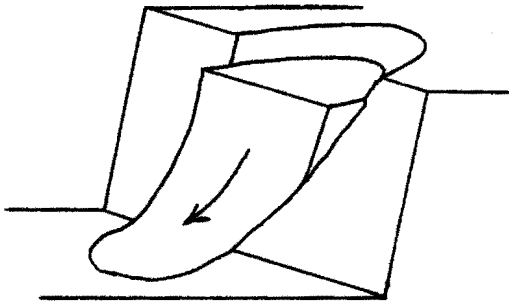


Figure 10. Slump.

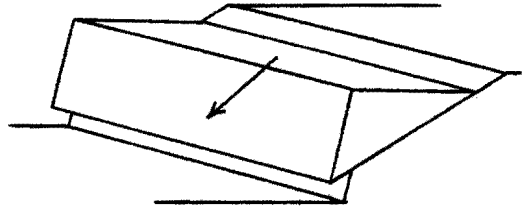


Figure 11. Plane.

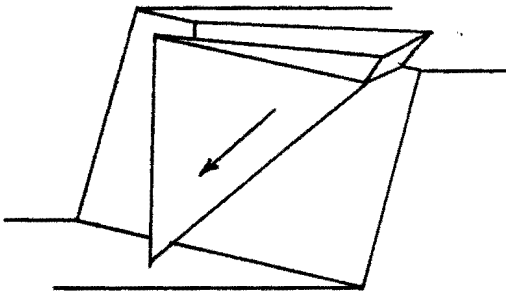


Figure 12a. Wedge,
sliding on 2 planes.

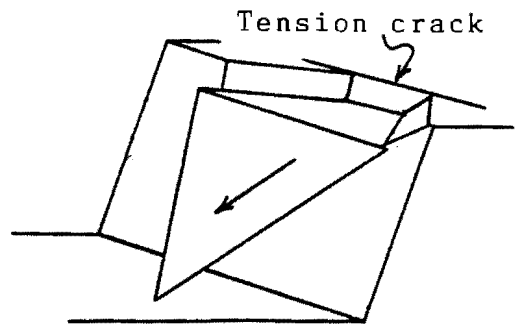


Figure 12b. Wedge,
with tension crack.

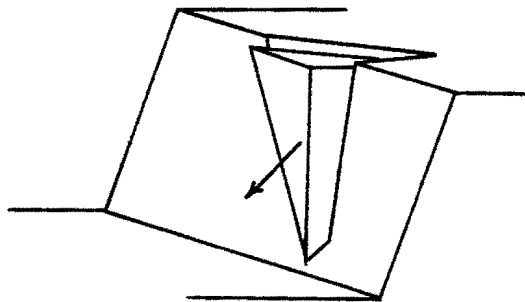


Figure 12c. Wedge, sliding on 1 plane.

Wedge

The most common failure in rock slopes is by wedging, as shown in Figure 12a, and is caused by the intersection of discontinuities of different attitudes. The criteria for this type of failure is generally as follows:

1. - The line of intersection of the two planes must daylight in the slope.
2. - Both planes must exhibit unstable conditions.

Variations in the wedge failure model include:

1. - There can be a tension crack which strikes sub parallel to the strike of the slope, and which may collect and hold water, adding to the downslope sliding force, as shown in Figure 12b.
2. - The orientation of the two discontinuities may be such that sliding occurs only on one plane. The other plane and/or any additional tension cracks could further increase the downslope sliding force because of trapped water, shown in Figure 12c.

Analysis of wedge failures involves extensive mathematical calculations with numerous variables. Interacting forces between the planes must be considered. The use of the computer is certainly advantageous in this type of analysis.

Topple

Sliding failures, such as planar and wedge, can be analyzed using to the planar characteristics of cohesion and internal angle of friction, but conditions may exist such that the dimensions of the block and inclination of the failure plane cause toppling instead of sliding.

The following relationships of θ , ϕ , b and h , as shown in Figure 13, describe how toppling and other simple sliding failures can occur:

1. - Stable condition: $\theta < \phi$, $\tan \theta < b/h$.
2. - Sliding only: $\theta > \phi$, $\tan \theta < b/h$.
3. - Toppling only: $\theta < \phi$, $\tan \theta > b/h$.
4. - Toppling and sliding: $\theta > \phi$, $\tan \theta > b/h$.

As viewed in cross section, the weight vector must fall outside of the base of the block in order to allow toppling. Failure by toppling combined with sliding can be very complex, involving the interactions of numerous various sized blocks, as shown in Figure 14.

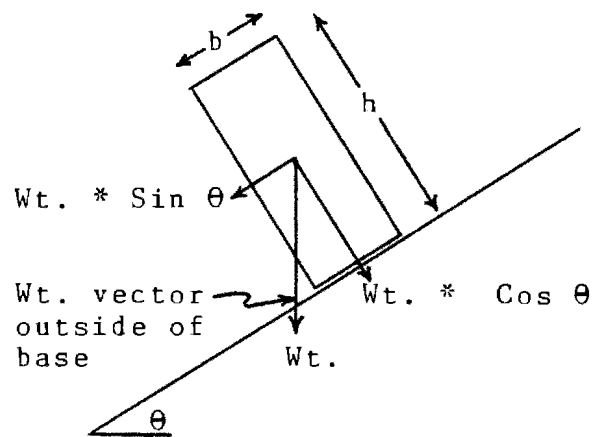


Figure 13. Simple toppling.

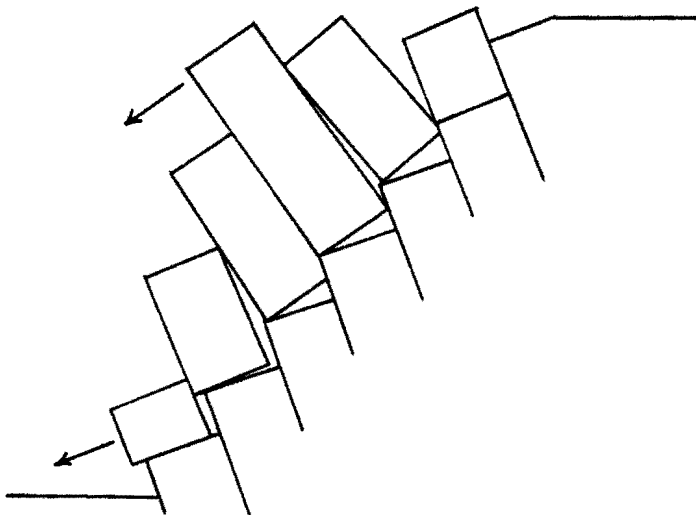


Figure 14. Complex toppling and sliding.

CHAPTER III

METHOD

As stated earlier, this report is concerned primarily with the line mapping method of gathering geological data on rock slopes. The method was developed by Piteau and Associates, Inc. (Piteau and Martin 1977) as a guide to field geologists, and has been used by numerous government agencies and private contractors concerned with rock fall. An example of a data sheet normally used to record field data is shown in Figure 15. This recording method is being replaced by hand held calculators programmed to receive and store large amounts of coded data, which can then be fed directly into a larger office computer for data manipulation and analysis (Watts and West 1985).

EXPLANATION OF LINE MAPPING

The line mapping method requires coordination of data gathering and standard geological mapping. Computer analysis is meaningless without the ability to locate the safe and hazardous areas on a map for future use.

The study area needs to be brushed as clear as possible, and strategic spads or nails implanted for reference purposes in the rock slope for the length of the

SITE:		COORDS. OR COUNTRYSIDE										NAME:					
		NORTH				ELEV.		TREND		OBS.							
TRAVERSE STATION						EAST		LOCAT.		PUNGE		LENGTH	DATE:				
STATION (FROM)	IRAV IREND	DISTANCE (FEET)	ROCK TYPE	STR TYPE	NO. JHS	JN SPCG (FT)	STRIKE	DIP		SIZE LENGTH (FT)	INFILLINGS		WAINNESS		REMARKS		
								+ ANG	DIR		INDEX	WR TIG	ILA	WAVE LITE			
1	5	0	12	17	19	21	23	26	29	31	34	35	38	41	42	46	

Figure 15. Field data sheet.

slope, such that each pair of reference points are separated by less than the length of one's tape, and that the tape stretched between each pair of points will be suspended for its entire length. These points should then be located by a survey crew so that the recorded geology can be put onto a scaled map.

At the top of the data sheet (see Figure 15) is room for all pertinent information concerning the origin of each tape line. Included is the name of the beginning control point, its coordinates in space, and the trend of the tape line from this point. The data is then recorded, using predetermined codes, in the appropriate columns as discussed in the Appendix.

Once the data have been collected, they must be analyzed to locate potential hazards, and to describe the conditions of instability. On slopes where few structural discontinuities are encountered, the individual potentially hazardous areas should be easy to identify. Tests to determine the internal angle of friction and cohesion will lead to solving Factor of Safety equations for each discontinuity. When numerous structures are present, the use of stereonetts and/or computer programs will be very helpful in determining stabilities of intersecting discontinuities which may not be initially apparent.

Although all structural discontinuities must be described, certain features do not lend themselves to the

detailed analyses of Hoek and Bray (1981). When a rock slope face has been cut along a joint, the hangingwall block has already been removed. Using Piteau's method, one would record a major, steeply dipping discontinuity, but it actually presents no hazard. Therefore it must be noted that this joint is the slope face. Should the strike of the road cut or pit wall deviate from the joint strike, hangingwall blocks may remain, and stability analysis will be necessary.

When the slope is blocky, with slabs on the order of one to two meters across, detailed stability analyses of each block would be very impractical. It would be sufficient to note the conditions, and to consider a bolt and net system for stabilization, as shown in Figure 16.

STEREONETS

An efficient method of displaying and manipulating field data is with a stereonet. (For full details on the general use of stereonets, consult Goodman (1976), Hoek and Bray (1981), or Ragan (1973)). The discontinuities may be plotted using one of three methods:

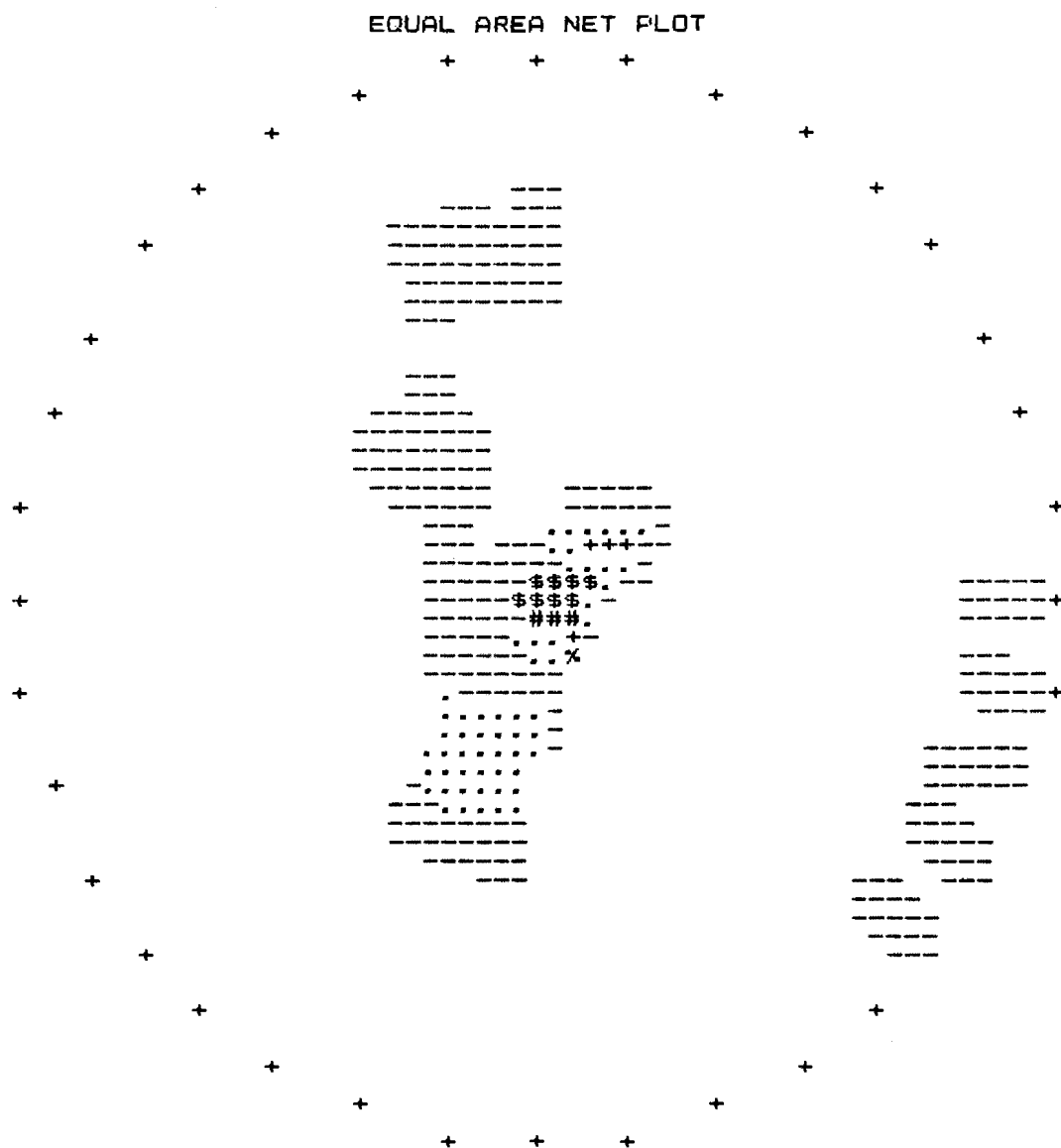
1. - Strike and dip.
2. - Normal pole to the plane.
3. - Dip vector.

The use of poles and dip vectors allows for more data to be plotted without cluttering up the stereonet. In the case of



Figure 16. Bolt and net stabilizing system.

plotting numerous joints or joint sets, the pole or dip vector concentrations may be contoured to indicate the highest density of orientations of the joints. This attitude set can then be used to represent the joint set in further calculations. Figure 17 shows a stereonet plot of joint dip vectors as produced by the computer. The numbers indicate the number of dip vectors which plot within one degree of that particular orientation. Figure 18 shows the contours of the values shown in Figure 17. One disadvantage to just blindly feeding data into a computer is shown in these two figures, as there are 10 dip vectors which plot at or near vertical. One cannot distinguish the different



TITLE: ELK1

CONTOURED PLOT OF DISCONTINUITY DIP VECTORS

Value Range	Symbols	Value Range	Symbols
0	Blank	7 to 10	"%"
1 to 2	"-"	11 to 14	"\$"
3 to 4	"."	14	"#"
5 to 6	"+"		

Figure 18. Stereonet of contoured dip vectors.

striking joints. Therefore, these discontinuities must be separated according to their strikes before plotting.

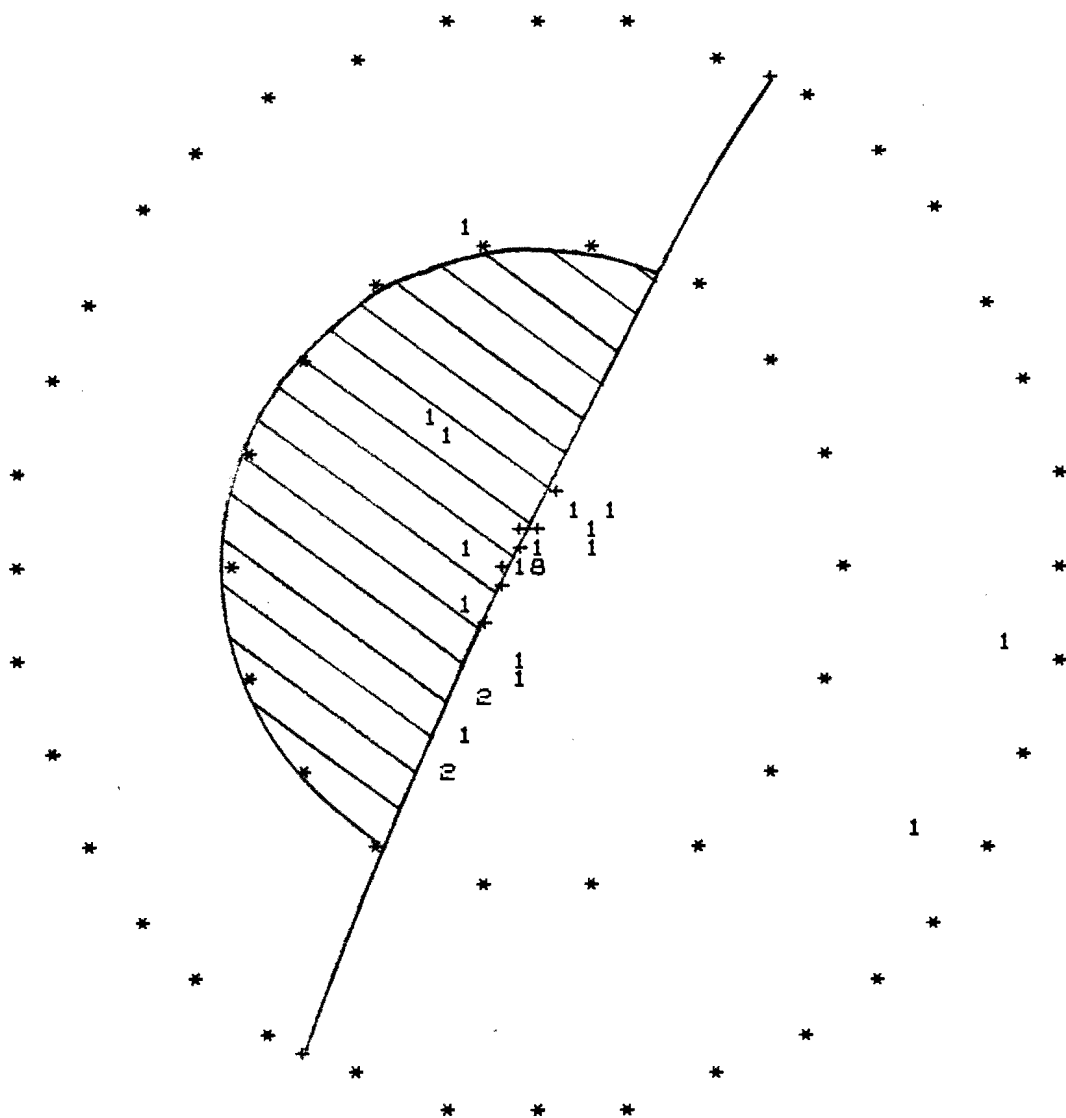
Markland Stability Test: Plane Failure

Variations of the usual use of stereonet, known as the Markland Stability Tests (Hoek and Bray 1981), allow for the rapid determination of potential failures. Dip vector plots are used to analyze for potential plane failures by the first of the two Markland tests. In addition to the discontinuities, a circle representing the internal angle of friction is plotted, as well as the great circle representing the rock slope face. Figure 19 is a computer printout of the Plane Failure test of the Elk 1 study site. The critical area of potential failure is the shaded area between the friction angle circle and the rock slope great circle on the downslope side. Any discontinuities which plot in this zone dip less than the slope face and more than the angle of friction. For simple plane failure, the discontinuities must also strike within 30 degrees of the rock slope face, or else they require release planes. Should the discontinuities plot on the updip side of the slope face, failure by toppling may result. These individual structures can then be field checked if necessary, and analyzed for stability.

Markland Stability Tests: Wedge Failure

The Markland Wedge Failure test is similar to the

MARKLAND STABILITY PLOT
PLANE FAILURE



FILE(S): / ELK1.DAT

of stations = 28

PHI= 41 SLOPE DIP DIR= 297 SLOPE DIP= 86

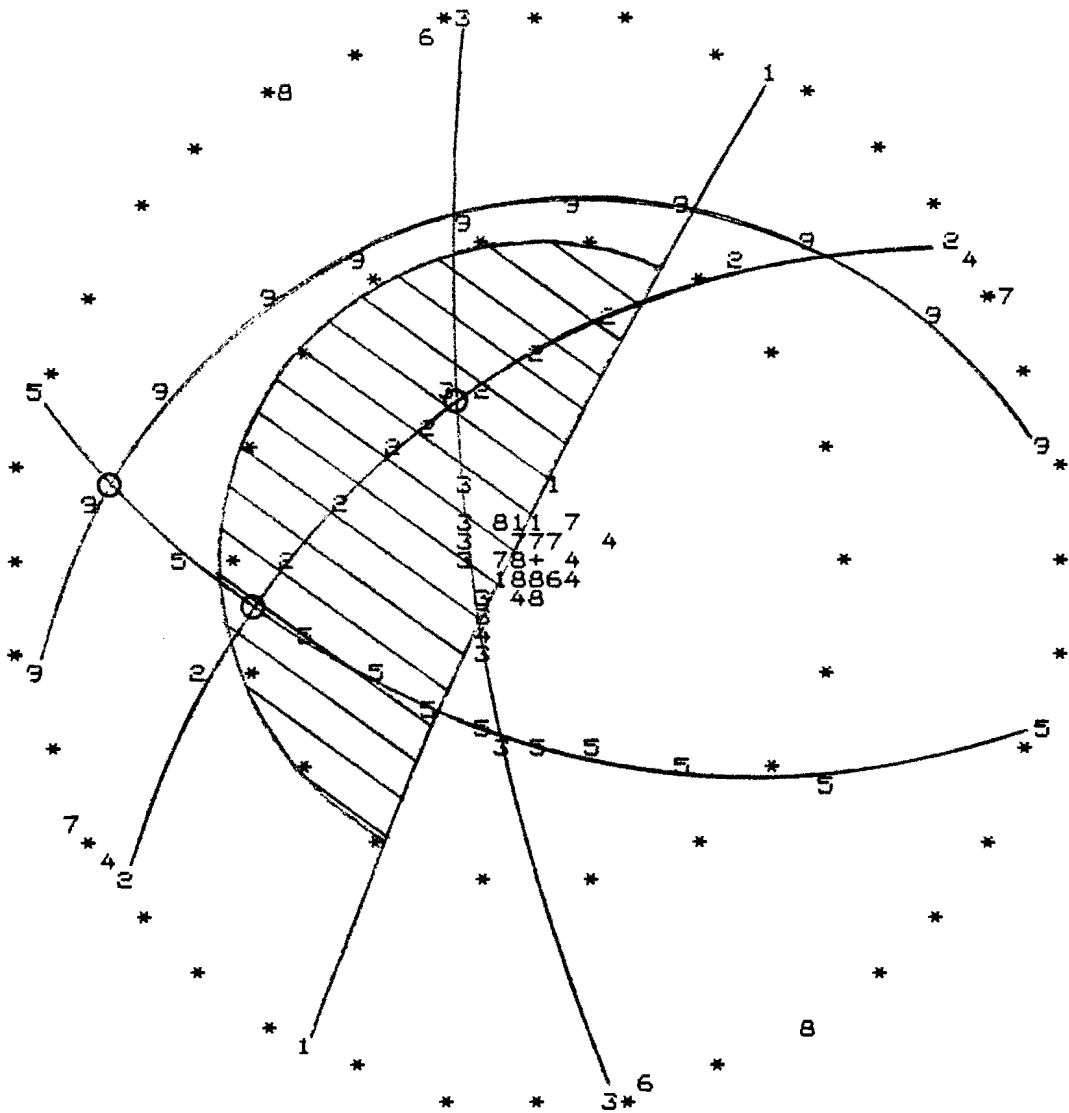
Figure 19. Stereonet of Markland Plane Failure Test.

plane failure test in that it uses the circle of the internal angle of friction and the great circle of the rock slope face. The discontinuities are plotted as strikes and dips instead of dip vectors. The same critical area exists as in plane failure, and any great circle intersections in this area are potentially hazardous. Figure 20 shows a computer printout of the Markland Wedge Failure test of the Elk 1 study site. Great circle number 1 is the rock slope face, and discontinuities 2 and 5 intersect almost directly on the circle of internal friction, indicating the potential for failure along these two planes. Planes 5 and 9 intersect outside of this friction circle, and should therefore be stable.

The spatial relationship of the intersecting planes must be considered before further calculations are performed. Two structures may intersect on the stereonet, but may actually exist at opposite ends of a particular study area, and may not intersect in physical space, as planes 2 and 3 of Elk 1 in Figure 20. Also, the plunge of an intersection may not daylight above the base of the slope, making the wedge far more stable than the calculations may indicate.

An additional aspect must be considered when mapping around potential wedges, or field checking computer predicted wedges; the possible existence of a steep to vertical dipping tension crack behind the wedge. Structural

MARKLAND STABILITY PLOT
WEDGE FAILURE



FILE(S) : ELK1.GRT

FILE(S)
FHI= 41

1

CU

our

43

47

407

8

3

[illegible]

Figure 20. Stereonet of Markland Wedge Failure Test.

orientations from mapping may indicate the presence of such a tension release plane, which can also add tremendous downslope forces on the wedge when filled with water. The exact location of a tension crack must be found for accurate force calculations to be performed.

When analyzing existing slabs and wedges which have little or no infilling material on the failure planes, those features which plot in the critical zones of the Markland tests will have Factors of Safety of less than 1.00. It must be noted that in both of these tests, any cohesion which may be present is not accounted for. There may actually be areas of high cohesion, and extremely rough or interlocking features on the failure planes which are not visible to the mapper. These conditions will contribute to a Factor of Safety greater than 1.00, which must be the case because the slabs and wedges have not yet failed. The intermittent presence of water pressures will also contribute to the ambiguity of Factor of Safety calculations. That these slabs exist is reason enough for further study and possible preventative maintenance.

COMPUTER SYSTEM

To handle the data, a computerized rock slope stability program (Watts 1986) was used. The program system allows for data storage and stereographic plotting, as shown

in Figures 17 through 20. It performs the extensive mathematical calculations necessary for Factor of Safety evaluations for plane and wedge failures. In addition, it will allow for back calculations of angle of friction, cohesion, and water pressure values which may have been involved in any failures. The water pressures reported are used as buoyancy pressures which can float the wedge resulting in loss of contact across either one or both intersecting planes during failure. When the buoyant pressures exceed the weight of the plane or wedge, a negative Factor of Safety is reported by the computer. The water "pressure" reported on the tension crack has units of kN or lbs, which is the water pressure multiplied by the tension crack surface area, and is the total downdip force added to the downdip weight of the wedge. Both pressures are converted to forces for the calculations done in the program.

The program system will also compute stabilizing rock bolt patterns and tensions, should the user wish to approach the engineering aspect of rock slope stability. As this study is of the geological aspect of data gathering and evaluation, the engineering aspects of this system were not needed.

CHAPTER IV

STUDY SITES

Two study sites were chosen to demonstrate the use of the line mapping system. Both are located within the boundaries of the Siskiyou National Forest of southwestern Oregon in the western Klamath mountains, as shown in Figure 21. The two study sites are the Powers site and the Elk River site.

REGIONAL GEOLOGY

The following discussion of the regional geological setting of the study area is based on the review of the geological studies of Dott (1971), Irwin (1977) and Ramp, Schlicker and Gray (1977).

Lithology

Rocks cropping out in this area range in age from Jurassic to present. Figure 22 shows the simplified geology and structure of the area, and also shows the geological setting of each study site. Table V has a brief description of the regional lithology.

Structure

The structures of the study area consist mostly of low

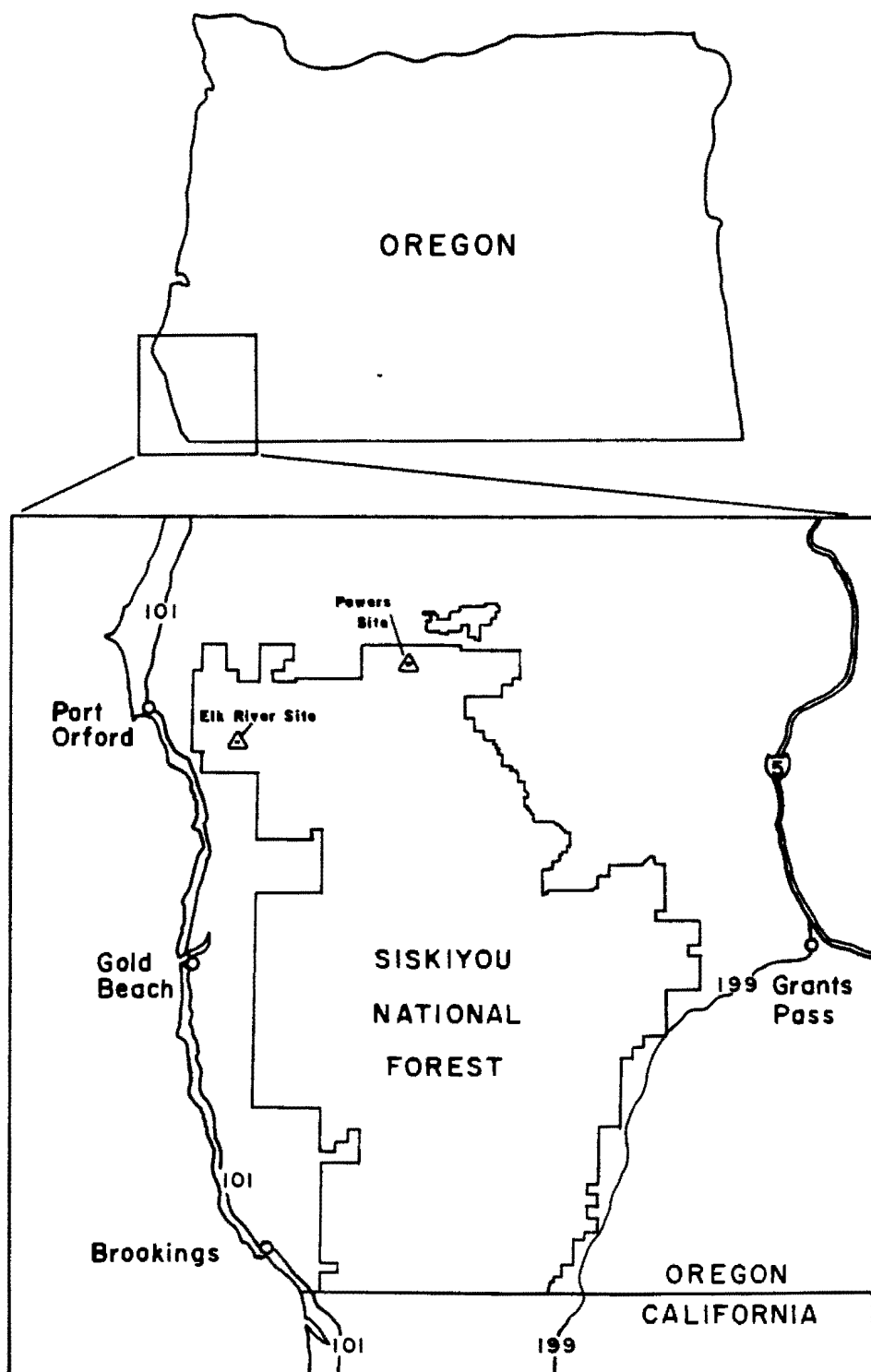


Figure 21. Study Location Map.

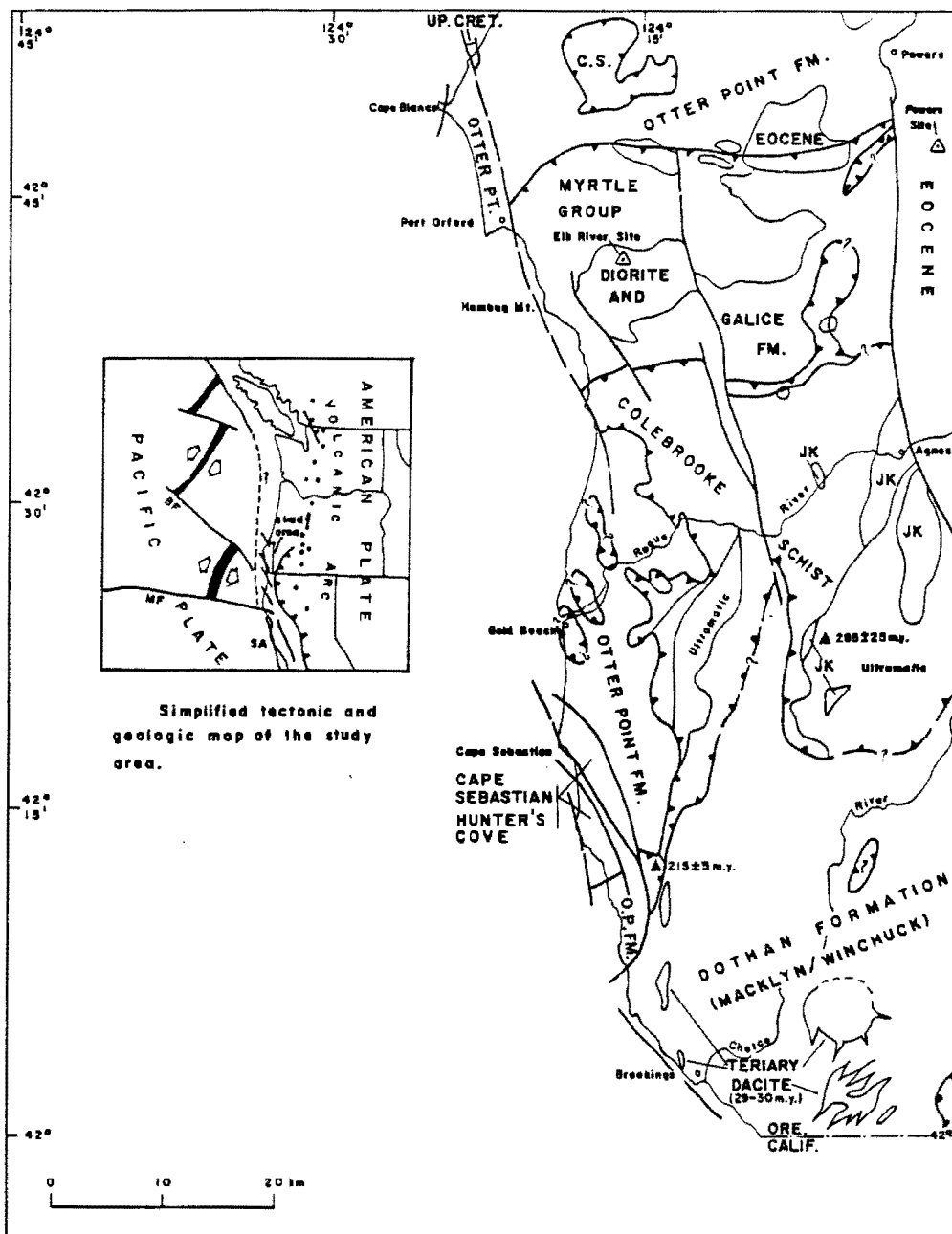


Figure 22. Simplified Regional Geological and Structural Map. (Dott 1971)

TABLE V
LITHOLOGY OF STUDY AREA
(DOTT 1971)

PERIOD	EPOCH	FORMATION	DESCRIPTION
Quaternary	Recent		Sands and gravels
	Miocene		Rhyolite and dacite intrusions
Tertiary	Oligocene	Tyee Formation	Massive sandstone and siltstone
	Eocene		
	Paleocene	Umpqua Group	Unconformable sequence of sandstone, shales and pillow basalt
Cretaceous		Hunter's cove	Thinly bedded sandstone
		Cape Sebastian	Massive sandstone
		Myrtle group	Humbug Mnt. conglomerate, Rocky Point sandstone
		Otter Point	Highly sheared sandstone
		Dothan Formation	5500 meter thick graywacke
			Diorite intrusion
Jurassic			Ophiolites
			Ultramaphics
		Colebrooke Schist	Quartz-mica phyllites, blueschist facies sandstones
		Galice Formation	Meta-carbonaceous argillite
		Rogue Formation	Meta-volcanic sequence

low angle thrust faults, dipping both east and west; probably extensions from northern California (Ramp, Schlicker and Gray 1977). Certain features are associated with these faults, as follows:

1. - Tectonic associated metamorphism and melange,
2. - Underthrusting (east dip) as seen by older rocks on top of younger,
3. - Dothan/Otter Point western plane is "an undulating blanket-like feature that becomes steeper and plunges at its line of emergence" (Ramp, Schlicker and Gray 1977), where it first crops out to the east,
4. - The major fault zones contain serpentinite bodies.

Following this sequence of thrust faulting was a sequence of high angle (near vertical) north trending dextral slip faulting, which offset the thrust faults, (see Figure 22). Examples include the Mountain Well fault, which separates the two main Colebrook schist bodies, and the Powers-Agnes fault, which separates the Tertiary and Cretaceous-Jurassic rocks north of Agnes, Oregon. The Powers-Agnes fault also offsets the east-west trending sinistral Canyonville fault just north of the study area, which itself apparently post dates the tectonic thrust faults (Perttu 1976).

All formations, with the exception of those of

Quaternary time, have undergone mild to intense folding, and include some overturned beds. Generally, the older the formation, the more intensely folded it is. Most fold axial planes trend north, but local east-west axial planes exist in the northern Cretaceous and Tertiary sediments.

Sequence of events

The Rogue and Galice Formations are interpreted as island arc volcanics and associated deep oceanic sediments in the basins behind the arc, with geosynclinal deposits near the continental coast. Due to Jurassic subduction and subsequent compression of the geosyncline and island arc, thrusting of this material against previously accreted material occurred, and these beds were folded and overturned. These volcanic and sedimentary deposits were apparently "rafted" on ultramafic rocks of oceanic crust/upper mantle origin, and the ultramafics and ophiolites are seen to crop out around the borders of these thrust sheets.

During late Jurassic time, dioritic intrusional activity occurred throughout the area, followed by the emplacement (obduction, Dott 1971) of the Colebrooke Schist with more associated ultramafic oceanic crustal material. According to Irwin (1977), there are no dioritic intrusions in the Colebrooke.

Erosion of these thrust sheets was an ongoing

process, resulting in "windows", and isolated sheets such as the Colebrooke and western ultramafics seen overlying the Otter Point Formation. There is some confusion as to the emplacement mechanism of the Colebrooke so far across the surface of the Dothan without more deformation of that underlying formation. Ramp, Schlicker and Gray (1977) maintain that the Dothan was thrust beneath the ultramafics, Colebrooke, Galice and Rogue, and that the Colebrooke is just more intensely metamorphosed Galice.

The area was then subjected to high angle faulting during Tertiary time, offsetting all formations older than Tertiary, with further folding of the coastal rocks. Large deltaic sandstone deposits were formed during Eocene time. An example of these is the Tyee Formation. The high angle faulting continued, possibly up to Quaternary time, as seen by the offsetting of Pliocene rocks (Ramp, Schlicker and Gray 1977). The Port Orford shear zone may still be active today (Irwin 1977).

The description of the regional geological and structural setting is not a crucial part of rock slope stability analysis. In this case it is included to indicate that this area is still tectonically active. This activity will contribute to the instability of the rock slopes.

POWERS

General

The Powers study site is located on National Forest Road 33, approximately nine kilometers south of the town of Powers at milepost 56.5, (see Figure 21). It is a 300 meter long section of road cut (Figure 23) in the Tyee Formation, consisting mostly of fine to medium grained sandstone. The rock is gray in color where fresh, but the exposed surface weathers to buff. Its bedding is locally near horizontal, but dips up to 15 degrees were recorded. The sandstone is interbedded with one to five meter thick beds of loadcasts (Figure 24) consisting of loosely packed and poorly cemented, layered, brown silty sandstone surrounding coarser gray spherical to ovoid sandstone cores. Periodically, these cores fall out, relatively intact, ranging in size up to one meter in diameter. One of these exposed loadcast beds underlies a section of the outcrop. This bed will erode at a faster rate than the overlying sandstone because it is considerably softer than the sandstone. Consequently, the sandstone will eventually be undercut, possibly causing collapse and/or toppling failure.

The outcrop contains two major vertical joint sets; one nearly parallel with the slope face, the other intersecting the face at a high angle. There are also numerous joints and minor joint sets at various

orientations. There are no observed faults or shear zones.

The road follows the South Fork of the Coquille River, which forms vertical cliffs up to 100 meters in height. Most of these cliffs are back away from the road and across the river, and were not involved in local road construction



Figure 23. Powers road cut.

or widening. The study slope face, however, was drilled and shot along one of the vertical joint sets to heights up to 30 meters above the road during widening. The slope above this reaches another 500 meters in height at a slope of approximately 50 degrees. Figure 25 (insert) shows current elevations, contours and the cliff top outline.

Upon initial inspection of this site, several failures



Figure 24. Load casts at Powers.

and potential failures were noted, prompting the choice of this road cut for analysis. That rocks do indeed fall here and present a hazard to traffic is evidenced by the presence of fallen load casts, and can be seen by the sandstone embedded asphalt shown in Figure 26. Many of the joints in the cliff face are open, some exceeding ten centimeters in width.

Since this site is composed of one rock type, it was initially divided into segments strictly on the bases of the locations of the mapping control points. Data gathering began at point 2-01 and continued through 2-8A in a southward direction, and then from 2-01 north to 2-10

through 2-14, (refer to the topographical map, Figure 25). The general slope configurations are shown in Table VI. Each segment was mapped, and analyzed using the computer system described earlier, to reveal the potential hazards in each area. Included were any interacting structural features which might exist at the borders of each segment. Due to the extremely dense vegetation cover, it was not



Figure 26. Sandstone in asphalt at Powers.

TABLE VI
GENERAL SLOPE CONDITIONS
OF POWERS STUDY SITE

LOCATION	HEIGHT (Meters)	LENGTH (Meters)	SLOPE DIP	SLOPE DIP DIRECTION	UPPER SLOPE DIP
POWERS 1	25-35	61.0	85	266	50
POWERS 3	35-50	12.5	85	274	50
POWERS 4	20-50	59.1	74-90 *	273	50
POWERS 6	20-25	16.3	90	287	50
POWERS 8	20-30	36.0	84	274	50
POWERS 8A	15-20 **	26.5	84	274	50
POWERS 10	15-20	13.7	77	271	50
POWERS 11	15-35	17.1	71	280	50
POWERS 13	3-25	17.5	68	300	50

* 30 meter slope is overhung; dip = 80, dip dir. = 093.

** Tape line at top of 5 meter talus slope.

Note: Field determined ϕ = 46 degrees.

Rock unit weight = 24.7 kN/m³

possible to determine the existence or location of any tension cracks behind the potential failures.

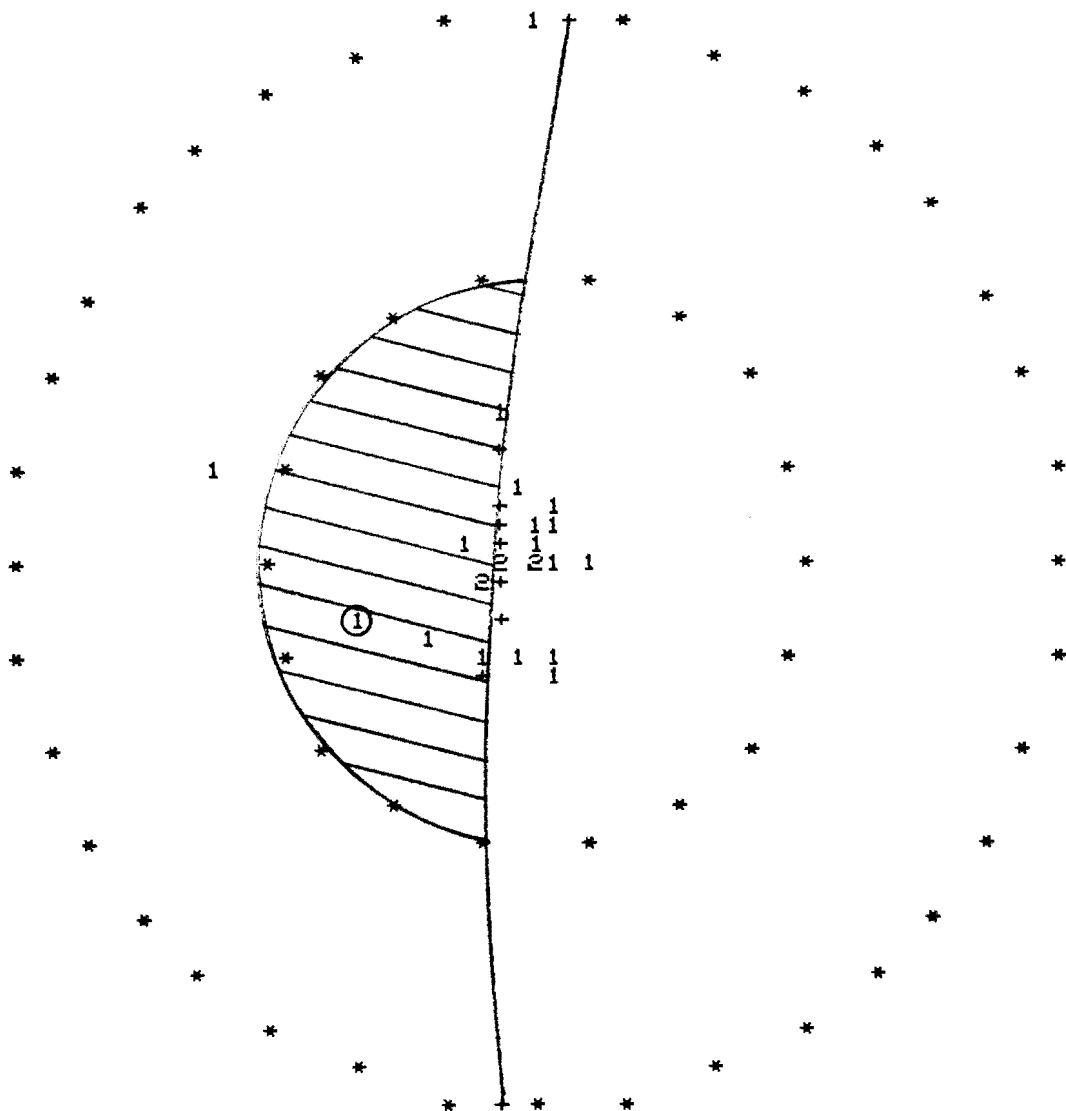
Procedure

Figure 27 shows the field data set for Powers 8A. The cut cliff face begins at the top of a five meter talus slope, and reaches a vertical height of 20 meters. The tape line was strung above the talus slope. The actual taped locations of the potential failures discussed below are given with all other potential failures in the results discussion later on in this chapter.

Initial visible inspection showed numerous one to two meter blocks formed by the intersecting joint sets, as well as exposed load cast centers near the top of the outcrop. The discontinuities responsible for these blocks were recorded, but much larger failure types involving discontinuities longer than three meters were analyzed for. All of the data was fed into the data file and plotted on stereonet using the Markland Plane test program, using the field determined internal angle of friction of 46 degrees, as displayed in Figure 28. Seven discontinuities are seen in the shaded critical area. These individual discontinuities were analyzed for their size and importance according to physical characteristics such as attitude, openness and previous movement. Those structures which appeared too small were neglected. Any cracks that extended

MARKLAND STABILITY PLOT

PLANE FAILURE



FILE(S): / POWERS8A.DAT

of stations = 23

PHI= 46 SLOPE DIP DIR= 274 SLOPE DIP= 84

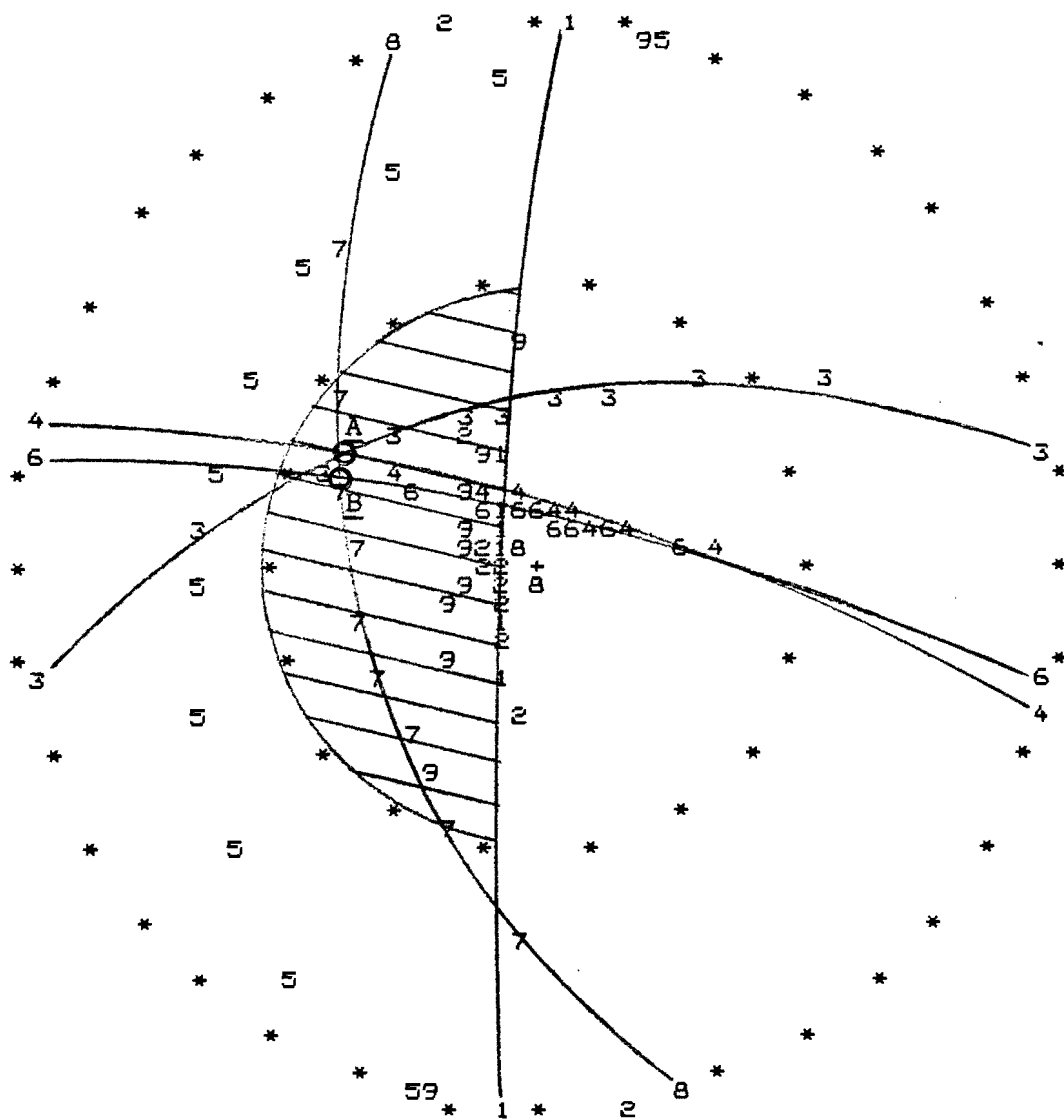
Figure 28. Powers 8A Markland Plane Failure Test.

less than three meters across the face and stopped in intact rock were considered stable. Also unused in this analysis were those structures labeled as slope face, having no hangingwall blocks present.

After applying the plane failure criteria discussed earlier, only one discontinuity (circled in Figure 28) presented any threat of failure. Since the slab is still in place, its Factor of Safety is at least 1.00, but the calculations resulted in a value of .55. Back calculations were then performed to determine what cohesive value the joint would need in order to maintain stability. When considered dry, this value was 36.9 kPa. When considered water saturated, the Factor of Safety dropped to -1.18 (buoyant pressures exceed the weight of the block), and the value of cohesion went up to 179.5 kPa, showing the effect water pressure has on stability. Because this structure shows evidence of water seepage, its cohesive value must be high. There may be some interlocking feature, not visible to the mapper, which contributes to the slab's stability. Since the plane strikes at 19 degrees from the slope face, it would probably require at least a weak zone to act as a release plane. With or without this release plane, this slab presents a potential hazard.

The structures longer than five meters were then plotted on stereonet as strikes and dips in the Markland Wedge test, as shown in Figure 29. Several great circle

MARKLAND STABILITY PLOT
WEDGE FAILURE



FILE(S) : POWERSBA.GRT

FILE (3)
PHI= 46

1

[illegible]

intersections occur within the shaded critical area, but after considering the spatial locations of each in the field, only two intersections presented any real hazard potential; planes 3 and 4, and 6 and 7. Each intersection is circled in Figure 29, and labeled A and B, respectively. Calculations for intersection A, assuming no cohesion and dry surfaces, resulted in a Factor of Safety of 2.38. By saturating the wedge planes for the worst possible water conditions, the water pressure on each plane reached 41 kPa, and the Factor of Safety dropped to 0.13, with contact maintained only on the shallower dipping plane. In order to maintain stability for this wedge, a cohesive value of 22 kPa is required. Calculations for intersection B resulted in a Factor of Safety of 1.12. With a water pressure of 41 kPa on each plane, contact would be maintained only on the shallower dipping plane, and the Factor of Safety would drop to 0.19. Cohesion of 130 kPa would be required to prevent this wedge from sliding. It must be noted that one plane of this wedge is the plane of the slab discussed in the plane failure analysis above. Essentially, this slab is cut by a plane to form a wedge at its northern end, and both the slab and the wedge would probably fail simultaneously.

Several joints and joint sets are near vertical, which can result in failure by toppling or collapse, provided that the horizontal base of these columns weathers out. In this section, these columns are less than one meter across and

are broken by the horizontal bedding, and do not present any more hazard than do the small blocks and load casts centers mentioned above, because they are all approximately the same size.

POWERS ANALYTICAL RESULTS

Following are the results from this analysis of the Powers study site. Only those features which present a potential hazards are discussed, and not every plane or wedge predicted by the computer system. Structure orientations are given as azimuths and dips, distances are referenced from each control point, which is also the name of each section. The cohesion necessary to maintain the stability of each hazard is also discussed, assuming the maximum water pressure possible and a Factor of Safety of 1.00.

Powers 8A

Powers 8A contains the small blocks and load casts seen throughout this road cut. In addition, there is a slab encountered at 26.5 meters, and which has a strike of 165 degrees, dipping 62 degrees southwest, towards highway 33. Its necessary cohesion for stability is 180 kPa. There are also two wedges, one of which daylights at .9 meters, the other projects to approximately 10 meters above the 25.3 meter distance. This latter wedge is actually part of the

plane encountered at 26.5 meters. The cohesive values of these two wedges are 22 kPa and 130 kPa, respectively.

Powers 8

Powers 8 contains two planes, at 2.1 meters and 24.4 meters. The first strikes at 163 degrees, and dips at 58 degrees southwest. Note the similar orientation of the slab of Powers 8A, indicating that this joint set is a prominent feature throughout this area. When considered under dry conditions, the Factor of Safety is .65, but when fully saturated, this value drops to -1.58. Under these saturated conditions, the slab's cohesive value is 357 kPa. The second potential failure strikes at 171 degrees, and dips at 65 degrees southwest. Its Factor of Safety drops from .48 to -1.11 when saturated, resulting in a cohesive value of 232 kPa to maintain stability.

One wedge exists at 1.8 meters, which is also a part of the first slab of this section. Its Factor of Safety drops from 1.05 dry to .13 wet, and its cohesive value is 50 kPa. A second wedge is indicated at 23.8 meters, but its orientation and location indicate that it would fail by toppling, if at all, because the intersection would daylight below the base of the cliff.

Powers 6

Powers 6 contains numerous one to two meter blocks, as well as some potential topplers. Several joint controlled

columns rest on the lower load cast bedding, but as yet show no outward movement. They appear to be stable at this time.

Powers 4

Powers 4 contains joint controlled columns 25 meters tall at the southern end of this section, shown in Figure 30. They rest on the very soft load cast bedding described earlier, shown in Figure 31. As can be seen, these columns



Figure 30. Powers 4 topplers.

have already moved, as evidenced by openings of approximately 10 centimeters at the top of the exposed joints. Should these columns fail, they would span highway 33, causing considerable damage.

Also included between the point P-04 and 13.4 meters, is a one meter thick by 12 meters tall slab, dipping at 74 degrees, which has already slid approximately 30 centimeters, and now rests on the ground at the base of the



Figure 31. Load cast base of Powers 4 topplers.

cliff. The load cast base projects beneath this area, but is not exposed. Presumably, this slab slid off its base, and is now leaning back against the cliff face. Because this failure was caused by the weak underlying bedding, a sliding failure analysis was not done. To compare this collapse failure with other unfailed slabs and wedges would not aid in the analysis of the stability characteristics of those unfailed hazards. The potential for hazard here is low, other than the blocks which rest on this already failed slab. No further serious weathering of the load cast bed beneath this should occur in the near future.

Powers 3

Powers 3 is a short section which contains one potential plane failure at 6.7 meters. It strikes 5 degrees and dips an average of 73 degrees northwest. Its dry Factor of Safety is .32, and its wet Factor of Safety is -1.36, resulting in a cohesive value is 290 kPa. Also included here is a joint intersection which results in a column perched on the near horizontal bedding. It is leaning into the cliff, and it poses no great threat.

Powers 1

Powers 1 area potential failures consist primarily of one to two meter blocks, and load cast cores. The jointing is very steep, with both major sets exhibiting curvilinear features, with dip values ranging from 40 to 70 degrees.

There are no major slabs or wedges here, but the load casts are the largest encountered at this study site, and present a potential hazard to traffic.

Powers 10

Powers 10 contains similar features to Powers 1, with the addition of two to three meter tall topplers located at 12.8 meters, which sit on the lower load cast bed.

Powers 11

Powers 11 contains a slab which projects through the nose of the curve of this road cut, as shown in Figure 32. It rests on the load cast bed because the potential failure plane does not penetrate through this bed. Its Factor of Safety when dry is .55, but this value drops to -1.97, resulting in a cohesive value of 177 kPa. Because of the proximity of the road to the slope, anything falling in this section will result in road damage. A second unmapped (above the tape line) failure plane is perched on top of this major slab. Because of the similar orientation to the major slab, its back calculated cohesive value would be the same, but its potential for failure is greater because it is not supported by the underlying bedding plane.

Powers 13

This last area is fairly stable, with the exception of the small joint controlled blocks. The height of the cliff

decreases to three meters at its northern end.



Figure 32. Slab through nose of curve at Powers 11.

POWERS CONCLUSION

The road cut, between Powers 1 and 8A was designed with a trench along its base to catch the small blocks and load cast cores which fall every year. Presently the trench is insufficient to prevent road and/or traffic damage. A small concrete retaining wall near the road should help to

clear this problem up. Northward from Powers 11, there is no road or traffic protection at all. Either netting and bolting this area, or widening of the roadway will be necessary to make this area safer.

Each of the major potential failures discussed would require an extensive bolting program to insure the future safety of the area. The exception to this would be the topplers discussed in Powers 4, which should be brought down under control, and the intact rock behind them should be bolted up.

ELK RIVER

General

The Elk River study site is located along Elk River on National Forest road 5325, approximately 25 kilometers east of its intersection with Highway 101. It is a 250 meter long section of road cut (Figure 33) in the Pearse Peak intrusion, consisting entirely of very hard, equigranular quartz diorite. When fresh, it is black and white, but it weathers to tan and buff. There are three prominent joint sets which intersect to form well defined wedges, several lesser joint sets and fractures, and two minor faults.

This site has been used as a rip rap source for road construction elsewhere, though it does not have the form of a quarry with flat pit floor and steep walls. The slope dips from 60 to 70 degrees, with some areas near vertical,

and with heights of 27 meters above the road. Above the cliff top, the natural slope dip is approximately 50 degrees to heights of 500 meters above the road. Whether the form of the slope is by construction or natural failings is uncertain, as there are no maintenance records. According to the local log truck drivers who use this road often, the site is in constant need of cleanup of debris on the road.



Figure 33. Elk River road cut.

Initial inspection of this site revealed several large blocks, 10 meters across, resting on inclined fault planes, and an area of falling debris containing boulders up to two meters in diameter. At no place is there a catch trench to collect the blocks as they fall, and they can roll onto and

across the road uninhibited.

ELK RIVER RESULTS

The Elk River site was divided according to the control points. The general slope configurations of each section are shown in Table VII. Data gathering began at the southern end, at point E-01, and progressed northward. The analytical procedure described for Powers 8A was employed, and the results follow. Refer to Figure 34 (insert) for the locations of the potential failures. As in Powers, all structural orientations are given in azimuths and dips.

Elk 1

Elk 1 contains one major plane at 4.9 meters, oriented at 51 degrees, dipping 62 degrees northwest, and is shown in the center of Figure 35, dipping to the left. It strikes within 24 degrees of the slope, and has a vertical release plane at 14.6 meters, striking 168 degrees, exposed in the upper left corner of Figure 35. Its Factor of Safety is calculated at .46 when dry, and -.42 when wet (buoyant pressures exceed the weight of the block), and its cohesive value under saturated conditions is 75 kPa. In addition, there are numerous blocks up to one meter across, some of which can also be seen in Figure 35. There are no wedges formed in this segment.

TABLE VII
GENERAL SLOPE CONDITIONS OF
ELK RIVER STUDY SITE

LOCATION	HEIGHT (Meters)	LENGTH (Meters)	SLOPE DIP	SLOPE DIP DIRECTION	UPPER SLOPE DIP
ELK 1	11	21.3	86	297	45
ELK 2	11-30	39.3	61	302	47
ELK 3	15-31	30.5	76	288	47
ELK 4	17-25	29.1	65	296	50
ELK 6	25-32	39.0	61 *	330	50
ELK 8	15 **	36.0	90	338	50
ELK 10	10 **	6.4	66	336	50

* Upper 18 meters is vertical.

** Tape line at top of 7 meter talus slope.

Note: Field determined $\phi = 41$ degrees.

Rock unit weight = 26.7 kN/m³

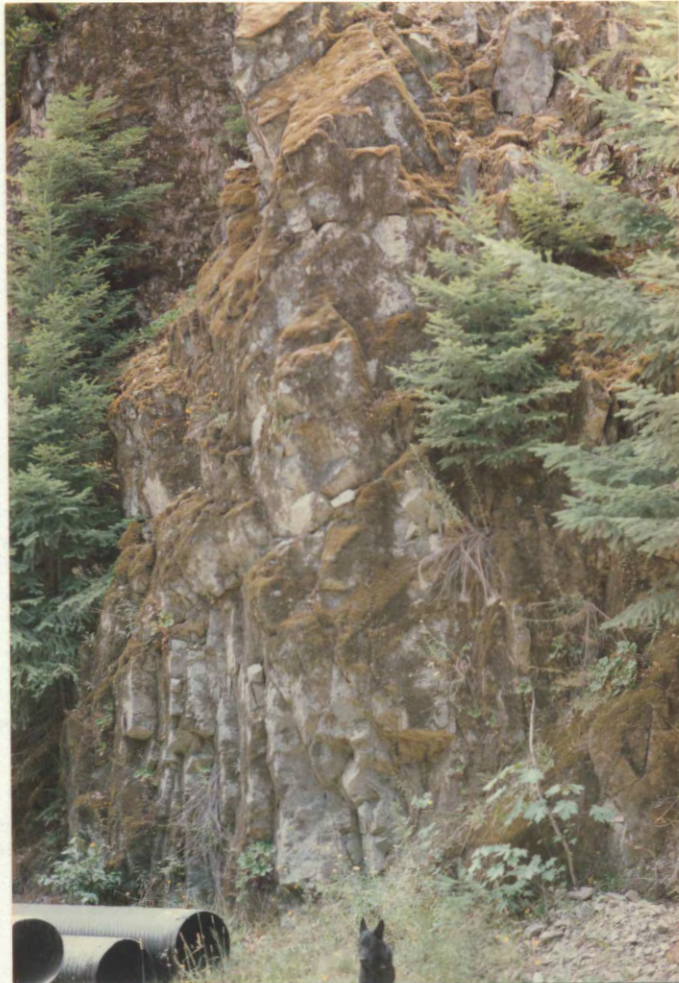


Figure 35. Potential plane failure at Elk 1, looking north.

Elk 2

Elk 2 contains no potential plane failures, but it does have a wedge whose intersection daylights approximately 13 meters above the road at the 8 meter distance, as shown in the center of Figure 36. It is bound by a joint seen at 10.4 meters, and a fault at 16.8 meters, whose internal

angle of friction is estimated at 22 degrees according to ϕ ranges for this type of discontinuity, as listed in Hoek and Bray (1981). A major joint system exposed in this outcrop acts as a tension release plane behind the wedge, oriented at due north, dipping 67 degrees west. With maximum water conditions, water pressures on each plane could reach 29.3 kPa, causing a loss of contact on the fault plane due to



Figure 36. Potential wedge failure at Elk 2.

buoyancy, and the water force acting on the tension crack could exceed 3978 kN. Under these conditions, the Factor of Safety is .78, and a minimum cohesive value of 70 kPa would be required to maintain stability. The wedges that existed on this fault below this point, have been removed, though

it is uncertain whether they failed or were brought down.

Elk 3

The first of two faults in this outcrop cuts through Elk 3, and has a curvilinear characteristic, with dips ranging from 40 to 74 degrees northwest. Its strike is 151 degrees. Due to its curved nature, the 40 degree (shallower) dip will control stability. Perched on this surface are two slabs, approximately four meters above the road; at 7.0 meters, shown in Figure 37, and at 14.0 meters, shown just to the right of center in Figure 38. As in Elk 2, the internal angle of friction for this fault plane is estimated at 22 degrees. A vertical joint set, mapped at Elk 1 and striking at 30 degrees, can be seen up the slope face, and will act as the release plane for both of these slabs. When dry, the Factor of Safety for the first slab is 1.04, and 1.07 for the second slab. When fully saturated, these values drop to -.30 and -.09, respectively. The cohesive value for the first slab is 184 kPa, and for the second is 168 kPa. As can be seen in Figures 37 and 38, these slabs are loose and have already moved a small amount, and are the most hazardous features of this outcrop. In addition to their conditions, should they fail, they would land directly on the road below.

Elk 4

Elk 4 contains no identifiable plane or wedge



Figure 37. First fault perched slab of Elk 3.

failures. The second fault of this outcrop, shown on the left in Figure 38, is located at the northern end of this section. Its strike is 35 degrees from that of the slope face, but there is apparently no release plane. The result of this orientation produces small blocks as seen throughout this outcrop.



Figure 38. Second fault perched slab of Elk 3.

Elk 6

Elk 6 contains one plane at 21.9 meters, oriented at 74 degrees, dipping 40 degrees northwest, which is within 14 degrees of the slope. It can be seen just to the right of center in Figure 39. When saturated, the Factor of Safety drops from 1.07 to $-.17$, and its cohesive value is 41 kPa.



Figure 39. Potential plane failure at Elk 6.

This segment also contains a very prominent wedge, shown in Figure 40, below. From initial inspections this wedge is failing, but apparently by small blocks, and not as one large wedge. It has a release tension crack exposed in the upper center of Figure 40. On the stereonet, the intersection plunges at only 10 degrees, indicating a very stable condition. With maximum water conditions, each plane could reach a pressure of 70 kPa, and the tension crack downslope force could exceed 16,000 kN, resulting in loss of contact on one of its planes. But due to its orientation, the Factor of Safety would drop from 2.63 when dry to .99



Figure 40. Major wedge at Elk 6.

when wet, and only 35 kPa of cohesion would be necessary to maintain stability. It looks much worse than it is. There is only the hazard of small pieces of it working out and falling. Its present condition could be due to its use as a rip rap source, but there are no production records to substantiate this.

Elk 8

There are no potential plane failures at Elk 8. The joint sets that intersect to form the wedge at Elk 6 also persist here, and form a wedge in the overhung cliff face at a slightly different orientation. The results of water on

this system are 56 kPa pressure on each plane, with 94,000 kN on the tension crack (due to its projected height). The Factor of Safety would decrease from 1.50 when dry to .82 when wet, and a cohesive value of only 20 kPa will maintain stability. The trace of this wedge is shown in Figure 41, with the daylighting intersection just to the left of center.



Figure 41. Trace of wedge at Elk 8.

This section also contains some large blocks, up to four meters tall, which perch on one of the more prominent joints, shown in Figure 42. These present a hazard to the road because they lean out at this time, and as they move and topple off the edge, they will fall directly onto the

road.



Figure 42. Topplers at Elk 8, looking north.

Elk 10

There are no potential plane or wedge failures in the Elk 10 segment. There are numerous small blocks which fall periodically onto the road, shown at the far end of the exposed outcrop in Figure 43.

ELK RIVER CONCLUSION

The construction of Forest Service road 5325 did not allow for any catch trench between it and the outcrop. There does not appear to have been any room for a trench between the outcrop and Elk River. Most of the hazard will



Figure 43. Elk 10, looking north.

be from small blocks working out of the slope, and falling or rolling to and across the road. The exceptions are the two loose slabs discussed in Elk 3. Due to the highly fractured nature of the slabs, bolting appears to be impractical, yet bringing the slabs down would cause extensive damage to the recently paved road. It may be possible to contain them with a bolt and net system.

CHAPTER V

CONCLUSION

During the course of this study, most of the aspects of the line mapping method of Piteau and Martin (1977) were found to be extremely useful. Generally, this method resulted in a good accumulation of data for detailed stability analyses. It forced the close scrutiny of every crack and joint in the outcrops, and lead to the discovery of several slabs and wedges which were not immediately apparent. The use of the data sheets and codes discussed allowed for a more rapid method of recording the data, and this system could be considerably faster with the use of hand held calculators programmed to receive such data.

The computer system of Watts (1986) was very compatible with the data gathering method, and would be helpful to any rock slope stability analysis. In addition to the rapid plotting of discontinuities on a stereonet for display and study, the availability of the Markland test programs proved crucial to the study.

Several precautions mentioned earlier are worth repeating here. To begin with, the main concept of falling slabs and wedges must be kept in mind during all data gathering. The field data sheets leave little space for

remarks such as noting the field observed intersection of structures, and noting that the slope face may also be the dip face of a prominent joint which is harmless in itself, as it has no hangingwall blocks which could fall.

Next, when working with a dip vector plot of numerous joints on a stereonet, it must be noted that near vertical joints of widely varying strikes will all plot together near the center, and a contour of these will reveal a high concentration of joints, erroneously representing one joint set. These joints must be separated by their strikes onto separate stereonets before contouring, so that each joint set may be represented by the contour concentrations.

When conducting the Markland test for wedge failure, it must be ensured that the intersections revealed in the test are actual or potential intersections in the field. Much work calculating stability factors would be a waste of time on structures which never meet in the slope face.

During the initial visual observations of these study sites, certain potential failures appeared far less stable than others. Specifically the two slabs at Elk 3, and the major wedge at Elk 6. Prior to line mapping and computer analysis, these would have been the primary targets for a maintenance program. After using the line mapping method and computer programs, the calculated instabilities of the slabs were found to be in agreement with the visual observations, but those of the wedge were in disagreement.

The rapid calculations performed by the computer for Factor of Safety analyses allowed for these conclusions. The reliability of these conclusions remains in question. Several years of observations at these sites would be required to prove or disprove the computer predicted stability conditions.

There is no substitute for experience when describing discontinuities with terms such as roughness, strength and ends (extent). Only past experience studying rock slopes could develop a good consistent feel for how these terms relate to stability. When a Markland test reveals potential hazards that should have already failed, then there must be some other stability factor holding the slab or wedge in place. It is difficult to relate a 2-dimensional view of a joint to the actual 3-dimensional conditions that exist all across the failure plane.

The back calculations for cohesion seem to offer a partial solution to the problem of having actual stability in the field but instability in the calculations. The mathematical formulas include conditions of internal angle of friction, water pressure, rock density and Factor of Safety, as well as cohesion, but they do not include the ambiguous conditions such as roughness and continuity across the failure plane. When the factor of safety is held at 1.00 (it must be at least that for the potential failures to be currently stable), the water pressure allowed to go to

the worst possible conditions, and the internal angle of friction as determined by field experiment is used, the value of cohesion is then the only remaining variable. This cohesion term must include the conditions of roughness and continuity. Although not giving a true value, this approach can be used to rate the stabilities of the failure planes to each other at each study site, provided that the rock density and internal angle of friction do not change across the outcrop. One can at least begin a relative stability analysis within a homogeneous outcrop.

The potential failures at the Powers study site require much higher cohesive values to maintain stability than do the structures at Elk River. Due to the lower internal angle of friction at Powers, this is not unexpected. When considering a relative rating of each structure as to its potential hazard, it is not advisable to compare a potential hazard in one rock type to a failure plane in another rock type. A comparison between rock outcrops should include the statistics of failure and maintenance at each study site, in addition to the stability analyses.

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APPENDIX

DESCRIPTION OF DATA SHEET ENTRIES

Column 1: Station.

This is the name of the tape point of origin.

Column 5: Traverse, trend.

For additional control points, the tape orientation can be recorded here.

Column 8: Distance.

This is the distance from the control point to the geological feature, and can be recorded in either English or Metric.

Column 12: Rock type.

Any letter or number code convenient to the mapper can be used. The information is useful not only as documentation for geological presentation, but also as an aid in determining quantities such as the internal angle of friction and of cohesion for intact rocks, weathering rates, and strengths when compared to published, extensive laboratory experiment results. (See Tables III and IV in Chapter II).

Column 15: Rock hardness.

The codes used by Piteau (1977) are shown in Tables I and II.

Column 17: Structure type. (Piteau,1977)

The common structures to record are as follows:

(Taken directly from Piteau's manual, 1977).

Axial Plane (AP) - The plane joining the lines of maximum curvature on successive layers of a fold.

Axial planes are imaginary planes which define the shape of folds and do not represent any physical discontinuity in the rock mass.

Bedding (BG) - Regular layering in sedimentary rocks marking lithological contacts.

Cleavage (CV) - Closely spaced parallel surfaces of fissility in rock not parallel to lithological contacts.

Contact (CN) - Surface between two rock types, one or both of which is not sedimentary

Dike (DK) - A sheet-like body of igneous rock that cuts across the structure in adjacent older rocks which it entered while in a molten condition.

Fault (FL) - Surface of shear recognizable either by the displacement of another surface that crosses it or by striated slickensides on the surface.

Faults thus include all "shears". Faults can be classified by the direction of slip of the fault block which rests on the fault plane (the hanging wall block). Refer to slip and separation under type of lineation. For descriptive terms use

- fault breccia (FB), - slickensides (SK),
- striae (ST), - gouge (GO), - mylonite (MO),
- fault zone (FZ), etc.

Foliation (FN) - Surface parallel to compositional contacts in metamorphic rocks.

Gneissosity (GS) - Surface parallel to lithological layering in metamorphic rocks.

Joints (JN) - Fracture in rock mass along which there has been no identifiable displacement. For descriptive and/or analysis purposes it may prove advantageous to record the genetic type if known. Some of these which could be considered are

- tectonic joint (TJ), - bedding joint (BJ),
- columnar joint (CJ), and - sheet joint (SJ).

Joint sets (JS) - recognized set of joints, which have the same attitude and length. The spacing and frequency of these joints is recorded. For descriptive purposes if these joint sets are tending to be uniformly related they could be referred to as a joint system (JY), and when they persist over great areas we designate this

jointing as the regional joint pattern (RJ).

Schistosity (SC) - Surface of easy splitting in a metamorphic rock defined by the preferred orientation of metamorphic minerals.

Shear (SR) - Surface of shear without recognizable displacement. It can be recognized by slickensides, polished or slickness of the surface, or striations on the surface.

Sill (SL) - A tabular body of igneous rock that has been injected while molten between layers of sedimentary rocks, or along the foliation planes of metamorphic rocks.

Tension crack (TC) - An unnaturally developed tension feature which is open and planar in form; such features could be tension cracks at slope crests or naturally occurring discontinuities which have opened.

Unconformity (UC) - Eroded surface covered by sedimentary rock.

Vein (VN) - Fracture in rock with a filling apparently injected at the time the fracture formed.

Column 19: Number of joints.

When a defined joint set is encountered, it may be recorded as a zone thickness of uniform joint spacing, with the number of joints in the zone recorded here.

Column 21: Joint spacing.

The spacing of the joints referred to above is recorded here.

Column 23: Strike.

The strike of the discontinuity is recorded as an azimuth. (Note: the computer program used requires that a strike of due North be recorded as 360 degrees.)

Column 26: +,-.

This is an optional column to designate the dip direction relative to the strike of the structure:

- + if the dip is in a clockwise direction from strike,
- if the dip is in a counterclockwise direction from strike.

Column 29: Dip direction.

The direction of dip, using the Cardinal points, may be recorded here.

Note: It may be advisable to establish a convention for recording the discontinuity orientations as is often done when using bearings, restricting the values to only two quadrants. For example, record strikes as between 271 and 90 degrees, through 360, although the computer program does not require such a convention.

Column 31: Size, length and ends.

This is a record of the length of a discontinuity exposed in the rock face. The continuousness (ends) of the structure is recorded as follows in Column 34: (See Figure A1)

- 0 - If both ends of a joint are visible.
- 1 - If one end is visible, the other extends out of the rock face, either up or down.
- 2 - If both end extend out of the rock face.

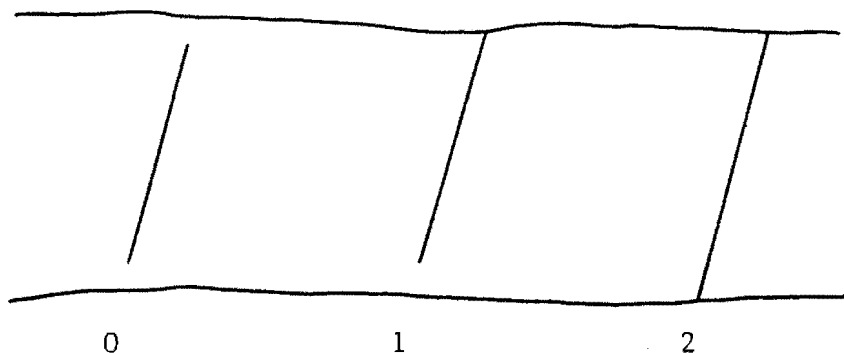


Figure 44. Ends of discontinuity.

Column 36: Infilling type.

This is a record of the material which may be present within a discontinuity, regardless of origin. Tests should be conducted to determine the shear strengths, c , and cohesion of this deposited material as it may act to hold the rock surfaces apart, thereby controlling slope stability. Only a few differing types of material are normally found within one slope, so that the codes used need not be too extensive.

Column 38: Infilling thickness.

Recording the thickness of the discontinuity, whether it contains infilling material or not, is done here, according to the following code system:

Category	Thickness	
	English (in)	Metric (cm)
1	0.00	0.0
2	0.00 - 0.25	0.0 - 0.5
3	0.25 - 1.00	0.5 - 2.5
4	1.00 - 2.00	2.5 - 5.0
5	2.00 - 4.00	5.0 - 10.0
6	>4.00	>10.0

Column 39: Infilling hardness.

The hardness (therefore unconfined compressive strength) of the infilling material is recorded here.

Column 41: Water.

Record water conditions here, according to the following code:

1. - The discontinuity is tight, no evidence of water flow.
2. - The discontinuity is dry, no evidence of water flow.
3. - The discontinuity is dry, but has rust stains or other water indicators.
4. - The discontinuity is damp, but has no free

flowing water.

5. - The discontinuity shows seepage, but still no free flowing water.

6. - The discontinuity has continuous flowing water.

It is important to visit the study sites through different seasons to observe water conditions.

Column 42: Roughness.

This is recorded according to the following system:

1. - Slickensided or polished.
2. - Smooth.
3. - Defined ridges.
4. - Small steps.
5. - Very rough.

Column 42 - 46: Waviness.

The waviness of a discontinuity may be recorded by its interlimb angle and the wavelength, as shown in Figure A2.

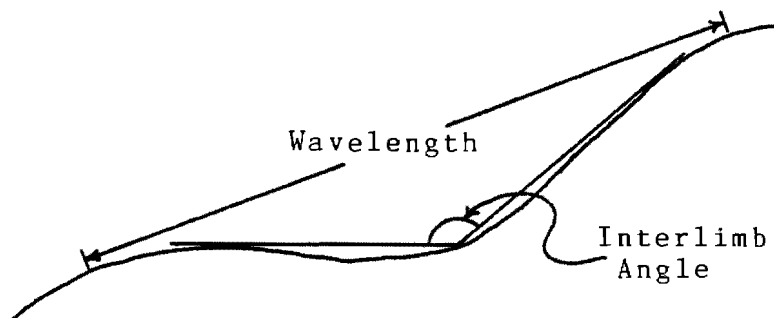


Figure 45. Waviness measurement.

The main computer program used in this study does not use all of the information gathered in the field. Data such as rock type, hardness, discontinuity type, and infilling material are of descriptive, but not mathematical, importance. Orientation, cohesion and friction angle are critical in limiting equilibrium analysis.